PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 4 ▽ JUNE, 1930

HIGHWAY DAMAGED BY GROUND FREEZING

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - - - - - - - See page 2 of cover for price

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 11, No. 4 **JUNE, 1930** **R. E. ROYALL, Editor**

TABLE OF CONTENTS

Page

Illustrations of Frost and Ice Phenomena 61

Progress Report of the Connecticut Avenue Experimental Road 69

Some Points of Contact Between Soil Science and Highway Engineering 77

THE BUREAU OF PUBLIC ROADS

Willard Building, Washington, D. C.

REGIONAL HEADQUARTERS

Mark Sheldon Building, San Francisco, Calif.

DISTRICT OFFICES

DISTRICT No. 1. Oregon, Washington, and Montana.
Box 3900 Portland, Oreg

DISTRICT No. 2. California, Arizona, and Nevada.
Mark Sheldon Building, San Francisco, Calif.

DISTRICT No. 3. Colorado, New Mexico, and Wyoming.
301 Customhouse Building, Denver, Colo.

DISTRICT No. 4. Minnesota, North Dakota, South Dakota, and Wisconsin.
410 Hamm Building, St. Paul, Minn.

DISTRICT No. 5. Iowa, Kansas, Missouri, and Nebraska.
8th Floor, Saunders-Kennedy Building, Omaha, Nebr.

DISTRICT No. 6. Arkansas, Louisiana, Oklahoma, and Texas.
1912 Fort Worth National Bank Building, Fort Worth, Tex.

DISTRICT No. 7. Illinois, Indiana, Kentucky, and Michigan.
South Chicago Post Office Bldg., Chicago, Ill.

DISTRICT No. 8. Alabama, Georgia, Florida, Mississippi, South Carolina, and Tennessee.
Box J, Montgomery, Ala.

DISTRICT No. 9. Connecticut, Maine, Massachusetts, New Hampshire, New Jersey, New York, Rhode Island, and Vermont.
Federal Building, Troy, N. Y.

DISTRICT No. 10. Delaware, Maryland, North Carolina, Ohio, Pennsylvania, Virginia, and West Virginia.
Willard Building, Washington, D. C.

DISTRICT No. 11. Alaska.
Goldstein Building, Juneau, Alaska.

DISTRICT No. 12. Idaho and Utah.
Fred J. Kiesel Building, Ogden, Utah.

Owing to the necessarily limited edition of this publication it will be impossible to distribute it free to any persons or institutions other than State and county officials actually engaged in planning or constructing public highways, instructors in highway engineering, and periodicals upon an exchange basis. Others desiring to obtain PUBLIC ROADS can do so by sending 10 cents for a single number or $1 per year (foreign subscription $1.50) to the Superintendent of Documents, United States Government Printing Office, Washington, D. C.

ILLUSTRATIONS OF FROST AND ICE PHENOMENA

Reported by IRA B. MULLIS, Associate Engineer of Tests, Division of Tests, United States Bureau of Public Roads

THE heaving of soils due to frost action and the subsequent loss of soil stability due to thaws constitutes one of the most important problems confronting both the highway and railroad engineer. The purpose of this report is to call attention to the many different ways in which frost action may manifest itself. As a basis for the discussion to follow, the physical laws controlling the performance of soils during frost action are presented in summarized form.

PHYSICAL LAWS CONTROLLING EXPANSION DUE TO FROST ACTION REVIEWED

Failures due to frost action may be due to physical phenomena occurring either individually or in combination with each other as follows: (a) The gradual expansion of freezing water; (b) the instantaneous freezing of supercooled water when the pressure productive of supercooling is removed; (c) the contraction and expansion of either ice or frozen soil due to temperature changes; and (d) the growth of ice layers in moist or wet freezing soils.

Expansion of freezing water.—Water at any temperature in excess of 4° C. (39.2° F.) expands when heated and contracts when cooled. It attains its maximum density at 4° C., however, and cooling below this temperature causes the water to expand. The rate of this expansion (*1*)[1] is such as to cause water at decreasing temperatures to possess relative volumes as follows: 4° C., 1.00000; 0° C., 1.00013; −5° C., 1.00070; −10° C., 1.00186. The corresponding densities are: 4° C., 1.00000; 0° C., 0.99987; −5° C., 0.99930; and −10° C., 0.99815. It is understood, of course, that unfrozen water at or below 0° C. can exist only when the water is under pressure.

The expansion which water undergoes on changing from the liquid to the solid state without change in temperature exceeds very appreciably any volume change due solely to change in the temperature of the water. This expansion amounts to 9 per cent of the initial volume (*2*). Thus 100 cubic feet of water at 0° C. may become 109 cubic feet of ice at the same temperature.

Freezing of supercooled water.—When the expansion of water due to cooling is prevented, the freezing point is lowered and the water exerts very high pressures. The interrelationship existing between the freezing point of water and the pressure under which the water exists, according to Bridgman (*3*), is shown graphically in Figure 1. According to this figure, supercooling to −4° C. causes the water to exert a pressure of about 6,500 pounds per square inch. This illustrates the tremendously high pressures exerted when supercooled water is not permitted to expand.

When the pressure exerted by the water exceeds the resistance of the container, freezing occurs at a rate dependent upon the speed at which the pressure is released. At the instant when solidification occurs the volume of the contained material increases 9 per cent.

Thus when the container consists of plastic materials such as thin lead pipe, partly frozen soil, etc., the release of pressure may occur gradually. But in a container formed of cast iron, ice, or solidly frozen soil, the pressure release will occur suddenly, and thus cause the ice to form in a more or less explosive manner.

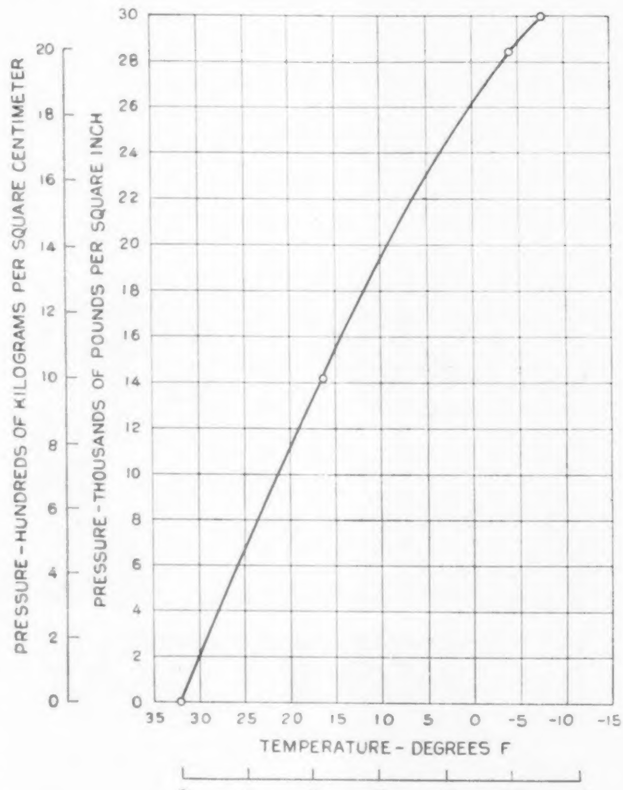


FIGURE 1.—RELATION BETWEEN THE FREEZING POINT OF WATER AND THE PRESSURE UNDER WHICH THE WATER EXISTS. DATA FROM SMITHSONIAN PHYSICAL TABLES (1921)

This explosive action of ice is illustrated by experiments by Major Williams (*4*) and described as follows:

> Having quite filled a 13-inch iron bombshell with water he firmly closed the touchhole with an iron plug weighing 3 pounds and exposed it in this state to the frost. After some time the iron plug was forced out with a loud explosion, and thrown to a distance of 415 feet, and a cylinder of ice 8 inches long issued from the opening. In another case the shell burst before the plug was driven out, and in this case a sheet of ice spread out all round the crack. It is probable that under the great pressure some of the water still remained liquid up to the time at which the resistance was overcome; that it then issued from the shell in a liquid state, but at a temperature below 0° C. (32° F.), and therefore instantly began to solidify when the pressure was removed and thus retained the shape of the orifice whence it issued.

The cake of ice furnished by J. L. Harrison, of this bureau, and shown from two different angles in Figure 2 also illustrates the instantaneous solidification of supercooled water. The water which formed this cake was but partly frozen in an alumium pan. When photographed the cake contained both free water and air. Due to the high degree of heat conductivity possessed by aluminum, freezing probably began simultaneously at the top, bottom, and sides of the water mass, thus inclosing the unfrozen water in an ice container. As the process of freezing continued, the contained water exerted increasing pressure until the resistance of the ice container was exceeded. The pressure being suddenly released, the supercooled water immediately solidified and the expansion due to freez-

[1] Figures in parentheses refer to reports listed in the bibliography at the end of this report.

ing caused the protruding "tooth" to be formed as shown.

The bulge in the top of the ice as shown in Figure 2 (bottom) is believed to be due to the expansion of solidifying water when confined by the surface crust of ice. This bulge was highest at the center and did not extend quite to the circumference of the cake. Fissures concentric with the circumference of the cake were observed along the circumference of the bulge in the bottom of the top crust of ice where cross bending was greatest. Apparently the supercooled water burst through one of these fissures to form the "tooth" or cone of ice shown.

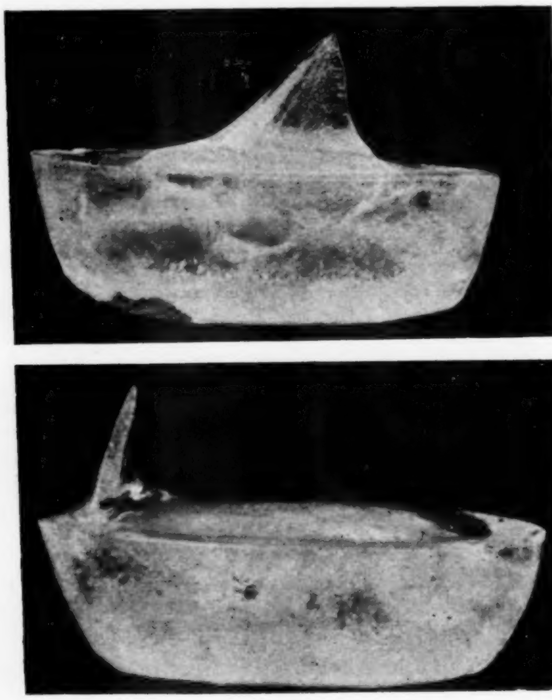


FIGURE 2.—A CAKE OF ICE WHICH FROZE ON ALL SIDES BUT BURST UNDER THE EXPANSIVE FORCE OF THE FREEZING WATER IN THE INTERIOR

Volume change due to temperature change occurring in ice.—According to tests performed by Petterson (*5*) the coefficient of linear expansion of ice is 0.000053 per degree centigrade. Therefore if the temperature of an ice cake 500 feet long were reduced from 0° C. to −18° C., its length would be reduced by 0.477 foot. Should this contraction result in cracks which are subsequently closed by newly formed ice, the length of the ice cake when returning to a temperature of 0° C. becomes 500.477 feet. With succeeding temperature alternations of this character, the length of the ice sheet would continue to increase. The effect of these temperature alternations on the growth of the ice is analogous to the strokes of a jack handle in moving loads. As the magnitude of the work done by the jack is governed by the length and the number of strokes of its handle, so is the growth of the ice controlled by the amplitude and the number of temperature variations below the freezing point of water.

Growth of ice layers in soil.—According to both Taber (*6*) and Bouyoucos (*7*) the formation of well-defined ice layers in freezing soil is due to three physical phenomena: (*a*) The ability of water particles in soil pores of comparatively large capillary dimension to freeze at or slightly below normal freezing temperature;

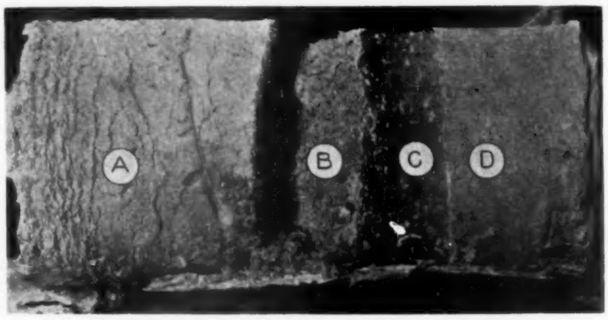


FIGURE 3.—FROZEN CLAY CYLINDER WITH SAND LAYER IN MIDDLE. A, FROZEN CLAY; B, SAND; C, ICE; D, UNFROZEN CLAY. FROM REPORT BY TABER (*6*)

(b) the ability of water particles in the finer capillaries to resist freezing until the temperature is reduced to a point below that at which the moisture froze in the larger capillary pores; and (c) the ability of water particles when freezing in the larger capillaries to draw to themselves unfrozen water from the finer capillary pores and thus increase in volume at the expense of the water furnished by the finer capillaries. Figure 3 illustrates the segregated ice layers in a cylinder frozen, by Taber.

Pressure effects due to frost action.—Both Taber and Bouyoucos state that enormous pressures are produced by ice crystals during growth. According to Taber the growing ice crystals may produce pressures in excess of 14 kilograms per square centimeter (199 pounds per square inch) and may cause water to be placed under a tension sufficient to lift a column of water over 150 meters (492 feet) in length.

The magnitude of the pressures exerted by expanding ice also is very high. The horizontal thrusts exerted upon dams and other structures due to this cause have been estimated to be as large as 34,000 to 47,000 pounds per lineal foot. Compressive tests (*5*) made on blocks of ice by Professor Brown of McGill University showed strengths as follows:

Temperature °F.	Crushing strength, pounds per square inch
28°	300
14°	693
2°	811

If the crushing strength of ice equals 400 pounds per square inch, as assumed by C. A. Mees in his paper on the design of dams (*5*), the thrusts exerted by various thicknesses of ice forming on the surfaces of lakes or similar bodies of water are as follows:

Ice thickness, inches	Horizontal thrust in pounds per lineal foot
6	28,800
8	38,400
10	48,000
12	57,600

But when the temperature of the ice approaches 0° F. the above values probably double.

VARIABLES FACTORS INFLUENCE THE MANIFESTATION OF FROST ACTION

Although the physical laws which control the volume increase of solidified water may be stated simply, the particular manner in which the effects of frost action are manifested depends upon a number of variables, among which are: Direction of heat radiation; size of soil particle; and amount of water available. Generally, the vertical manifestation of frost action is termed heave and the horizontal manifestation is termed thrust.

Direction of heat radiation.—According to Taber (*6*) the upward heave that accompanies the freezing of soils is due to the growth of ice crystals in a vertical direction, and this is determined by the direction in which heat is conducted away most rapidly and the availability of water necessary for growth. The importance of the direction of heat conduction is illustrated in the experiments described below.

Mixtures of white clay and water in different proportions were frozen in thin glass test tubes, half of them being buried in sand so that freezing was from the top down, while the others were exposed so that freezing took place from the sides inward. All of the latter were broken, longitudinal cracks extending the full length of the test tubes; but where frozen from top down none were broken, for the ice crystals grew only in a vertical direction.

FIGURE 4.—AN ICE HEAVE OR MOUND COMMON ON FROZEN RIVERS IN ALASKA (PHOTO BY U. S. G. S.)

Size of soil particle.—In coarse-grained sands free to drain no important frost heaves occur because practically all of the contained water freezes at normal freezing temperature and small unfrozen water particles do not exist in amounts sufficient to cause the frozen particles to suffer appreciable growth.

Permeable silts, which are capable of raising water rapidly and through considerable distances, are apt to suffer considerable frost heave.

The capillary tension may be higher in cohesive clays than in silts. The speed with which water rises in clays, however, is much less than in silts. Consequently, in dense clay soils with low ground-water level and absence of lateral seepage, only limited amounts of water are available for ice segregation. Under these conditions the soil adjacent to the growing ice crystals is apt to dry out and shrink, due to the loss of moisture. The ground-water elevation in clays must be comparatively high in order that much frost heave may occur or the clay must be wet, due to water absorption from the top of the ground.

Amount of water available.—According to Taber, high water content favors segregation and additional water may be drawn from the water table to form very thick ice layers. Studies made by Eakin (*8*) in Alaska indicated that—

> In materials which favor even distribution of water throughout the mass, heave is uniform over the entire surface and no differential vertical movement occurs. Thus, in fine, even-grained materials horizontal movement or movement with the surface is dominant, and even surfaces, either horizontal or sloping, result. On the other hand, irregular capacity for the retention of interstitial water leads to differential heave and thrust and to the development of surface irregularities.

These statements seem obvious, but differential heave also occurs in fine even-grained materials where the several areas receive quantities of water in varying

FIGURE 5.—SHOWING THE RELATIVE HEIGHT OF AN ICE HEAVE OR MOUND ON A FROZEN RIVER IN ALASKA. (PHOTO BY U. S. G. S.)

amounts just prior to the period of ground freezing. The differential heaving in this case, however, is not generally so pronounced as in soils possessing nonuniform capacities.

Furthermore, it should be remembered that when lakes or similar bodies of water are of such depth that water at the bottom is not reduced below 4° C. no freezing takes place at the surface even when the air temperature is considerably below 0° C. For this reason the surface of the deeper portions of deep bodies of water such as Hudson Bay and Lake Ontario have not been observed to freeze within historical time (*5*).

DETRIMENTAL FROST PHENOMENA ILLUSTRATED

The natural manifestations of frost action are necessarily varied in character because of the number of conditions under which the phenomena are apt to occur. Furthermore, the phenomena occurring in some complex combination instead of individually are probably responsible for detrimental pavement heaving or other damage. Under all conditions, however, the occurrence of these phenomena furnishes evidence of the enormous force exerted by freezing water. This is illustrated by the mounds, ramparts, frost boils, etc., referred to in the following discussion.

Figures 4 and 5 illustrate ice mounds familiar to those who travel in polar and subpolar regions when the rivers are frozen. The occurrence of these mounds is explained as follows:

With the beginning of freezing weather ice forms along the banks of the stream and becomes firmly attached to the soil and rocks located there. When the ice sheet becomes continuous from bank to bank and gradually grows thicker the flow channel becomes correspondingly smaller. Under these conditions the water beneath is apt to be compressed until the force caused by this compression causes the surface ice to heave at the weaker areas. Water frequently spouts through the fissures which often form about these heaves, and attendant flooding and freezing continues throughout the winter or until the volume of water is reduced to such amount that it may be contained beneath the ice. These pressures sometimes become so great that water is forced out through the banks between rock strata or other openings and quickly freezes.

Figure 6 shows a bank of soil composed of sod, bowlders and clay which, according to Doctor Buckley (*9*), was thrown up during the winter of 1898–99 by ice thrust on the edge of a lake in Wisconsin. The average dimensions of this bank were a height of about 4 feet, a breadth of base of about 11 feet, and a breadth of top

Courtesy Wisconsin Academy of Sciences, Arts and Letters

FIGURE 6.—THE RESULT OF ICE THRUSTS ON PICNIC POINT NEAR MADISON, WIS. (PHOTO BY DOCTOR E. R. BUCKLEY)

of about 4 feet. At one place this bank had a height of not less than 8 feet and had raised a tree of considerable size through this distance. (Fig. 6.) Doctor Buckley says that bowlders were in many places actually rammed into the bank in such a manner that they presented much the appearance of plums in a pudding. The bank, in many places vertical, was raised up and turned over by the ice shove and trees 12 inches or more in diameter were sometimes dislodged and moved.

The cause of ice ramparts has been quite clearly described by Gilbert (*10*) as follows:

> The ice on the surface of a lake expands while forming, so as to crowd its edge upon the shore. A further lowering of temperature produces contraction, and this ordinarily results in opening vertical fissures. These admit water from below and by the freezing of that water they are filled, so that when expansion follows a subsequent rise of temperature, the ice can not assume its original position. It consequently increases its total area and exerts a second thrust upon the shore. Where the shore is abrupt, the ice itself yields, either by crushing at the margins or by the formation of anticlines elsewhere; but if the shore is generally shelving, the margin of the ice is forced up the acclivity, and carries with it any bowlders or other loose material about which it may have frozen. A second lowering of the temperature does not withdraw the protruded ice margin, but initiates other cracks and leads to a repetition of the shoreward thrust. The process is repeated from time to time during the winter, but ceases with the melting of the ice in the spring.

OCCURRENCE OF FROST CRYSTALS AND FROST BOILS DISCUSSED

Figure 7 illustrates the occurrence of frost crystals and thin ice sheets frequently observed on clay roads which are more or less rutted. When the temperature of the ground surface reaches the freezing point, ice crystals form on the surface of moist soil, and ice forms on the surface of any pools of water which may exist

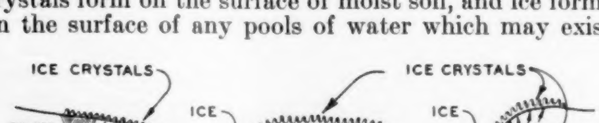


FIGURE 7.—ILLUSTRATING WATER MOVEMENTS, FROST AND ICE FORMATIONS ON A WET RUTTED CLAY SURFACE

on the surface. Following these freezing temperatures one frequently notices that these recently water-filled depressions are covered with thin sheets of ice with little or no water underneath. Upon further examination it will be noted that all fissures or other soil openings contain ice crystals. The absence of water underneath ice-covered depressions is due to the water having been withdrawn from under the ice sheets by the increased capillary tension of the soil produced by lowered temperature. It will be noted that ice crystals on the surface of clay soils will be longer than on those soils which contain less clay. Compacted clay soils will, however, contain fewer frost crystals at the surface than a similar soil in a less compacted state.

Should a layer of relatively dry sand exist somewhat below the surface as illustrated on the left side of Figure 7, the ice crystals are apt to be much shorter if present

FIGURE 8.—SHOWING THREE STAGES OF GROWTH OF ICE CRYSTALS FORMED ON THE SURFACE OF WET CLAY DURING THREE CONSECUTIVE NIGHTS OF FREEZING TEMPERATURE WITH LITTLE OR NO THAWING DURING THE INTERVENING DAYS

at all. It seems that the conditions essential for producing crystals of maximum length are found where the intensity of cold is such that the capillary movement is rapid enough to prevent freezing to an appreciable depth beneath the surface. This type of freezing is illustrated by Figure 8 which shows the growth of ice crystals during three consecutive nights when there was little or no thawing during the intervening days. Had there been a considerable drop in the air temperature in the early part of the third night of freezing, the height of the ice crystals in the lowest stratum (fig. 8) would have been proportionatly reduced and a frozen stratum of wet soil would have resulted.

Figure 9 shows the surface appearance of a "frost boil" under a surface-treated macadam road in New Hampshire. The origin of the water in the boil was traced to a leaking water pipe beneath the subgrade. The type of soil was a fine silt which served as a water pocket. Whether this heaving was produced by solid ice or by ice crystals which had become more or less segregated from the soil by frost action is not known. It seems probable, however, that much if not all of this heaving was due largely to a mixture of ice and soil of a form somewhat like that shown in Figure 8.

Figure 10 shows solid ice overlaid by a few inches of thawed material underlying a concrete pavement near Duluth, Minn. When first observed this frost heave must have extended 8 or 9 inches above the original grade line. The concrete pavement although heavily reinforced with longitudinal steel bars was badly shattered. An excavation made at one edge of the pavement disclosed that the pavement was laid on a bed of sand

Figure 9.—A Frost Boil in a Surface-Treated Macadam Laid on Bed of Silt. The Heave was Due to a Leaking Water Pipe Beneath Followed by Freezing

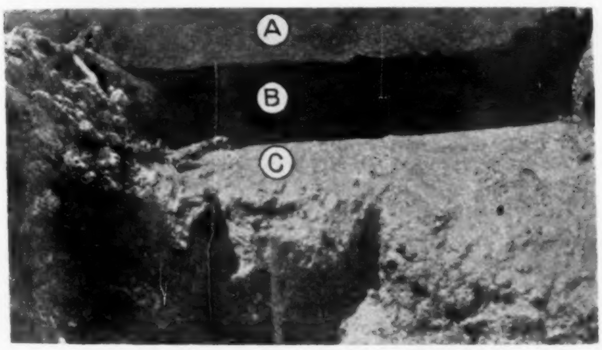


Figure 10.—Photograph of the Lower Part of a Ruptured Concrete Pavement at A; B, the Thawed Subgrade; C, the Underlying Prism of Solid Ice and Frozen Clay Underneath

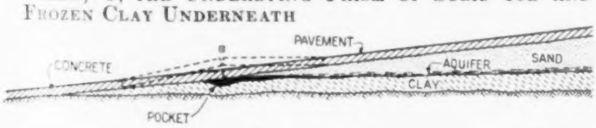
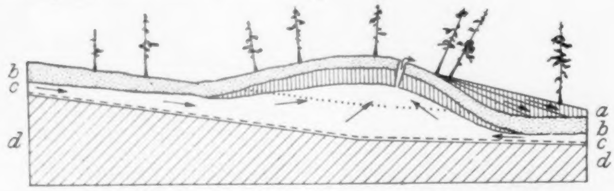


Figure 11.—Subgrade Profile Productive of Frost Heaves

underlaid by a stratum of clay. Where the dipping stratum of clay was intersected by the plane of the subgrade a water pocket formed as is illustrated in Figure 11. Clay shoulders in the sides and a formation of solid ice beneath (fig. 10) prevented the escape of the water from this pocket.

Figure 12 illustrates the occurrence of soil blisters. According to Nikiforoff, (*11*) these blisters attain enormous proportions in Arctic regions, extending more than 20 feet in height in Siberia. The method of formation is explained as follows: *d* represents ever-frozen subsoil which extends to considerable depths. The soil above this stratum thaws during the summer and as is characteristic of most Arctic regions is frequently quite wet, especially near the foot of slopes and in surface depressions where water collects. At the beginning of winter the surface soil, *b*, freezes down to the water in liquid form *c*. At this stage the liquid water is confined between two zones of hard-frozen material. When the pressure becomes sufficiently high to rupture the frozen surface soil, the supercooled water is forced through the crack and is probably frozen instantly. Figure 13 shows a tree split by a fissure occurring on the summit of an ice blister.

Figure 14 illustrates similar mounds formed of more or less segregated fragments and particles of alluvial materials believed to be of Pleistocene or recent age observed by Leffingwell (*12*) and others in Alaska. Most of these mounds are in the form of gentle domes ranging in height from less than 25 feet to a maximum of about 200 feet above the plain. Generally they have rounded tops and slopes of less than 15° from the horizontal but a very few are steepsided and have an angular break at a somewhat level top. Some of the more recent mounds contain craters which furnish fresh water to such extent during the summer that they may overflow.



Courtesy Soil Science

Figure 12.—Cross-Section of a Soil "Blister" in Siberia

Courtesy Soil Science

Figure 13.—Tamarack Tree on the Summit of a Blister, Split When the Blister Broke

Figure 14.—A Mound Formed of Mud and Gravel Believed to Have Formed from Springs Subjected to Frost Action. (Photo by U. S. G. S.)

FIGURE 15.—A FROST-HEAVED AREA AT THE FOOT OF A SLOPE IN THE LAKE CLARK-KUSKUSKOKWIM REGION, ALASKA, AND A CLOSE UP OF ONE OF THE FROST HEAVES. (PHOTOS BY U. S. G. S.)

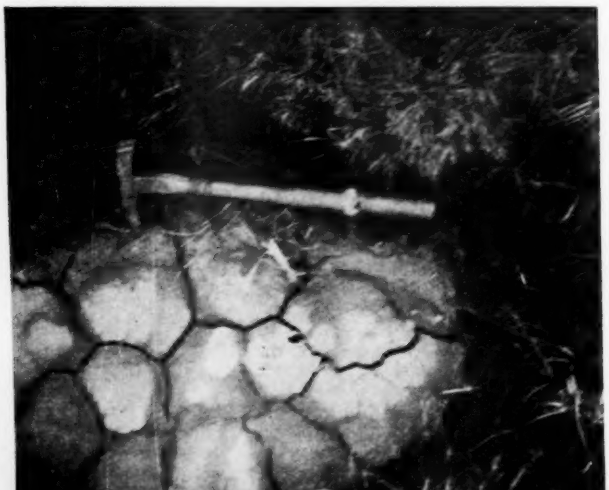


FIGURE 16.—A DRIED AND CRACKED MUD BOIL FOUND IN ALASKA BY EAKIN. (PHOTO BY U. S. G. S.)

Figure 15 (upper picture) shows a frost-heave area in the foreground at the foot of a slope in Alaska. These mounds, while small in comparison to some of those just described, seem to occur under conditions more or less similar to those productive of the larger ones. Where the development has progressed beyond the youthful stage these frost boils are invariably largely made up of

FIGURE 17.—A MUD BOIL IN A WATER-BOUND MACADAM LAID ON A FINE SANDY CLAY SOIL

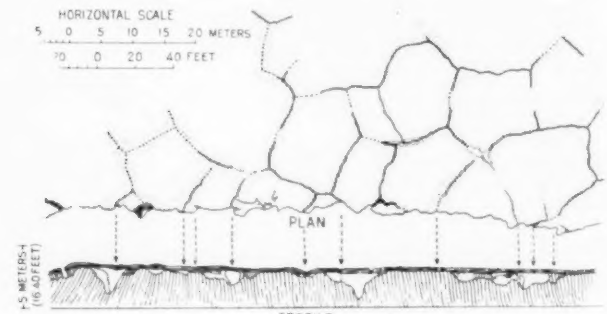


FIGURE 18.—MAP OF FROST CRACKS ON THE TUNDRA, WITH A SKETCH OF THE EXPOSURES OF GROUND ICE IN THE BANK AT ONE EDGE OF THE MAPPED AREA. (FROM REPORT OF U. S. G. S.)

mud at the center with the particles increasing in size toward the edge where stone and gravel predominate.

Figure 15 (lower picture) is a close-up view of one of these heaves. While geologists do not seem to be in full accord as to the exact manner in which this segregation of the finer particles from the larger rock fragments occurs, they are agreed that in all cases these mounds occur only where the ground is more or less thoroughly saturated with water and is subjected to conditions of alternate freezing and thawing of the surface layers of the ground (*8*, *13*).

Figure 16 shows the appearance of a dried and cracked mud boil found in Alaska by Eakin (*7*), which probably originated under the type of frost action just described.

The only difference between frost and mud boils is the manner in which the water accumulates. In reality discharging frost boils are nothing more than mud boils formed under frost action. Figure 17 shows a mud boil found about the middle of summer in the District of Columbia on the surface of a water-bound macadam laid on a fine sandy clay soil. This mud boil originated from a leaking water pipe and is similar in detail to the frost boils already described. It was first observed as a small mud-filled fissure on the road surface. With each passing load an eruption occurred and the volume of mud and water at the surface increased until a considerable mound was formed. Such boils may occur in any region where very fine particles of soil become reduced to the liquid state and under pressure the viscous fluid is extruded through fissures or other openings of such size as will permit its passage.

Figure 18 illustrates fissures formed when frozen soil in Alaska contracts due to a lowering of the temperature. According to Leffingwell (*12*) this cracking

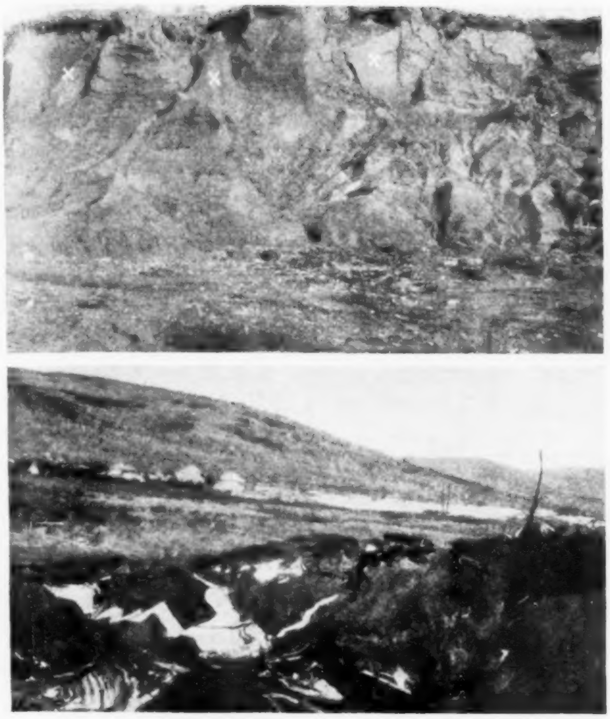


FIGURE 19.—ICE WEDGES IN FROZEN SOIL IN ALASKA. (PHOTO BY U. S. G. S.)

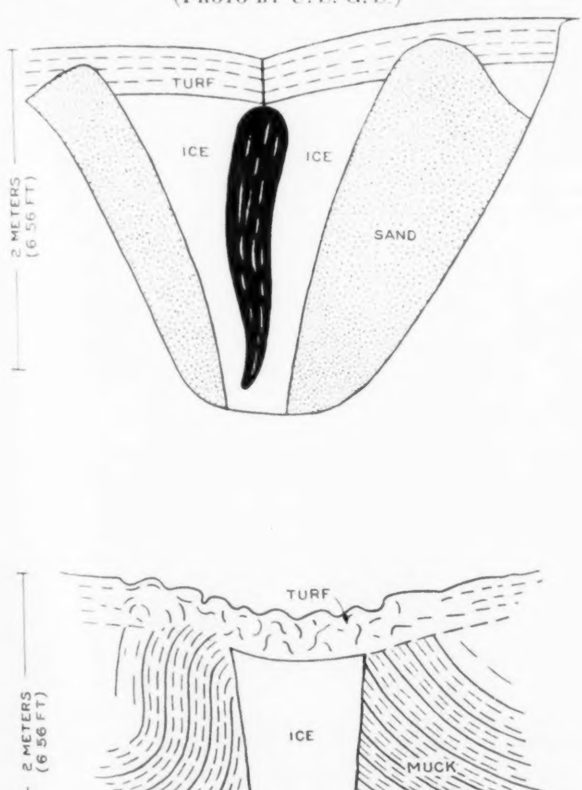


FIGURE 20.—ICE WEDGE IN SAND (UPPER PICTURE), SHOWING TUNNEL CUT BY DRAINAGE OF WATER THROUGH FROST CRACK, AND IN A MUCK BED (LOWER PICTURE), SHOWING UPTURNED STRATA. (FROM REPORT OF U.S.G.S.)

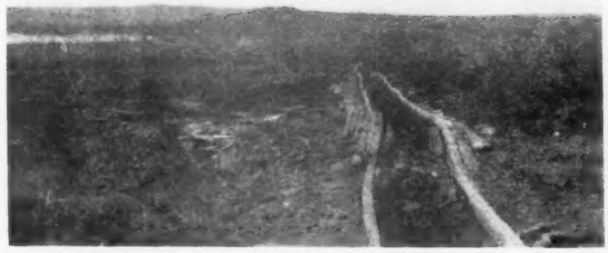


FIGURE 21.—EFFECT OF HEAVE AND THRUST ON THE POSITION OF A TRAM LINE IN ALASKA. (PHOTO BY U. S. G. S.)

FIGURE 22.—SHRINKAGE FISSURES IN FROZEN CLAY SOIL IN THE DISTRICT OF COLUMBIA

is frequently accompanied by loud reports and shocks sufficient in intensity to rattle dishes in camps. The individual blocks formed by these cracks have an estimated average diameter of about 16 yards. When the snow melts in the summer these cracks can be seen cutting across all tundra formations and even across the mud and the growing moss beds. Somewhat similar phenomenon has also been observed in Siberia by Nikiforoff (*11*).

Figure 19 illustrates ice wedges shown by × which are apt to form due to the cracking described above. The snow which fills the newly formed fissures soon becomes compacted and may reach a density equal to that of solid ice under conditions of alternate freezing and thawing such as frequently occur toward the end of the polar winters.

The disturbance of the adjoining soil due to formation of ice wedges is illustrated in Figure 20. When these wedges occur along the banks of streams and lakes large masses of earth are frequently displaced under wave or current action, causing the banks to have a saw-tooth appearance. The displacement of the tram line illustrated in Figure 21 is attributed at least in part to the formation of ice wedges.

Figure 22 illustrates fissures observed in frozen clay soil in the District of Columbia. Similar fissures ranging in width from one-eighth to one-fourth of an inch were observed also in Minnesota. In both cases the fissures contained hoar frost, but no ice crystals were observed. Hoar frost is always derived from water vapor but ice crystals such as those shown in Figure 8 are always formed from unfrozen moisture at the base of the crystals.

Figure 23 illustrates the type of soil migration which may occur when snowfall is heavy and frost penetrates the ground to an appreciable depth.

FIGURE 23.—AN AVALANCHE OF MUD, SOIL, AND ROCKS WHICH HAS SLID DOWN A HILLSIDE OF SHALEY SOIL

FIGURE 24.—EFFECT OF FROST ON SOIL MIGRATION WHERE THE PARTICLES RANGE FROM FINE TO COARSE AND ARE NOT COMPACT IN ARRANGEMENT

During the summer in polar and subpolar regions the surface of the frozen earth, which has become more or less thoroughly broken up under frost action during the long winter, is in a state favorable to becoming thoroughly soaked by water from melting snow and ice. Under this condition masses of viscous fluid of varying magnitude flow down most of the slopes and in certain cases fill the streams with great volumes of mud, ice, and water. Almost everywhere in polar regions where there are heavy deposits of snow, this type of soil movement is markedly present where long periods of alternate freezing and thawing occur (*14*, *15*).

Figure 24 shows the type of soil migration from the face of cuts in porous soils after frost action.

Differential heaving underneath road surfaces and pavements seems to occur when the subgrade soil possesses variable capacity for the retention of ground water. Coupled with this, there must be channels for furnishing water to the locations where the heaves occur.

Many investigators have reported the occurrence of frost heaves over water pockets similar to those shown in this paper. Arnold (*16*) found frost heaves in both cuts and fills under surface-treated macadam roads. Those in cuts were found to be due to seepage from contiguous slopes while those on fills were sometimes due to entrapped water flowing through the porous stone base from wet cuts at the upper ends of fills. Frost heaves may very readily occur on fills consisting of materials possessing both water capacity and permeability different in amounts.

According to Moffitt (*17*) most of the ice beds in Alaska occur either in the deposits of silt or between the beds of silt and the underlying gravel. Veins of ice in some places cut across the beds of silt and may form a considerable proportion of the silt deposits. The distribution of ice beds in many places is quite irregular and depends on conditions that are not understood. According to both Tyrrell (*18*) and Maddren (*19*), however, the ice originates from percolating water or from water under hydraulic pressure.

W. C. Buetow, State highway engineer, Wisconsin State Highway Commission (*20*), states:

> There are two kinds of boils, at least, in Wisconsin. The first appears—note I am not positive in this statement—to be caused by a live vein of water just under the subgrade. It usually appears on side hills. Troubles of this kind have in many instances been traced to a lake or swamp area which may be close at hand or miles away. The second type of boil, the kind that makes the most serious trouble, generally appears on the more level stretches of road and where the roadbed is built of a fine, medium clay, superimposed on a nonporous subsoil of exceptionally heavy clay or hardpan. The two types just described are the Wisconsin brand of boils. There may be others, but we are not familiar with them.

Differential frost heaving also may occur in deposits of silt or porous clay due to water raised by capillary tension from the ground water below. From what depths silts or clays at given densities may raise water in quantities harmful to the subgrade is not definitely known. Upon freezing, a stratum of soil ceases to receive water, but upon thawing a subsequent freeze may show a water content in excess of that found during the previous freeze. In contrast a wet frozen stratum of soil in contact with water beneath may become quite dry at the surface under conditions favorable to evaporation.

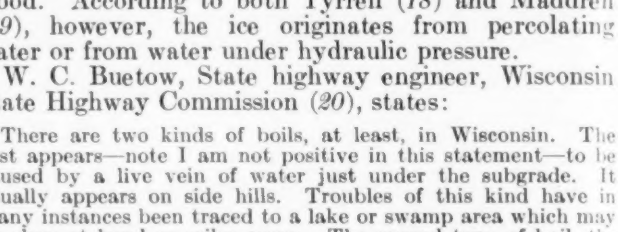


FIGURE 25.—ILLUSTRATION OF A ROAD PROFILE CONTAINING WATER POCKETS WHICH MAY PRODUCE FROST HEAVES DURING THE WINTER

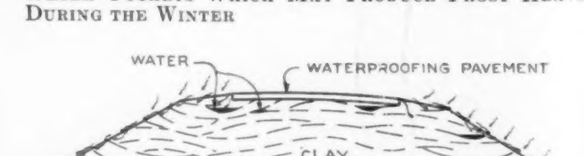


FIGURE 26.—ILLUSTRATION OF WATER POCKETS DUE TO IRREGULAR CAPACITY OF A FILL FOR RETAINING PORE WATER

CONCLUSIONS

It is apparent that the forces developed by action of freezing temperatures on water are enormous and may prove highly detrimental to highways and other engineering structures. From the foregoing discussion it will be seen that in order to minimize the destructive

(Continued on page 79)

PROGRESS REPORT ON THE CONNECTICUT AVENUE EXPERIMENTAL ROAD

MAINTENANCE AND BEHAVIOR OF SECTIONS DURING 1928 AND 1929

Reported by PAUL F. CRITZ, Associate Highway Engineer, and J. H. ELDRIDGE, Superintendent of Road Construction, Division of Tests, United States Bureau of Public Roads

THE Connecticut Avenue experimental road extending from Chevy Chase Circle to Chevy Chase Lake in Montgomery County, Md., was built during the years 1911, 1912, and 1913. The history of this project from the time of construction to 1928 is given in Public Roads, volume 9, No. 3, May, 1928.[1]

[1] Reports describing the construction and early behavior of these experiments are included in Circulars 98 and 99, Office of Public Roads; U. S. Department of Agriculture Bulletins 105, 257, 407, and 586, and Office of the Secretary Circular 77.

This report covers the maintenance and behavior of the various sections during 1928 and 1929. Figure 1 shows the location of the various sections, and the construction and maintenance costs are given in Table 1. Accumulated maintenance costs and traffic are shown in Figure 2. Analyses and quantities of materials used in construction are given in Tables 2 to 7, inclusive. The cost of past surface retreatments is included in Table 1, but the amounts of materials used have been

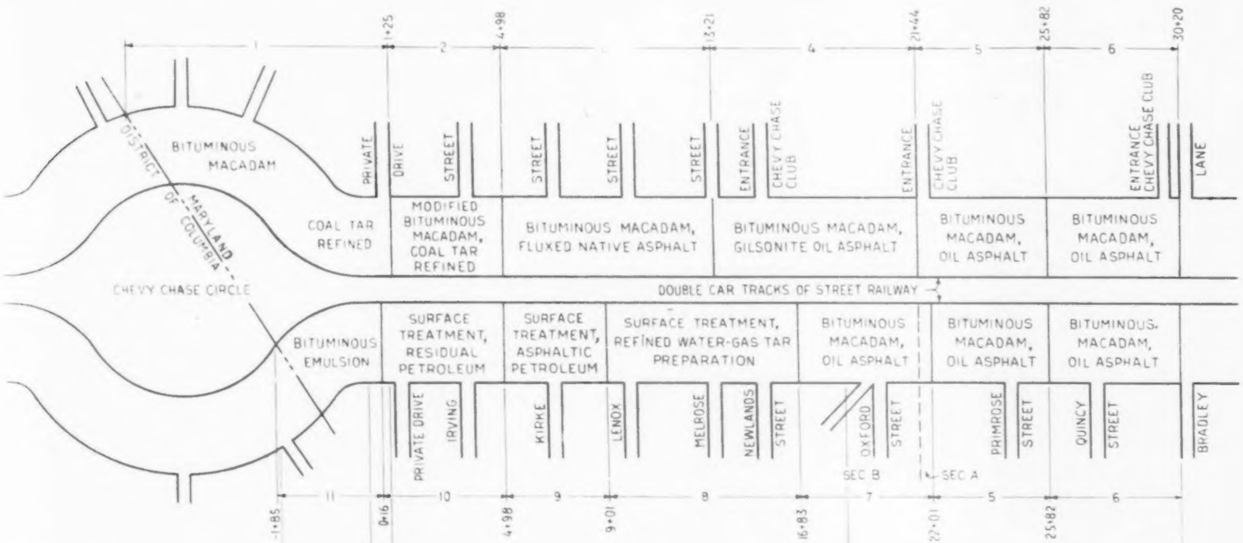


FIGURE 1.—LOCATION OF VARIOUS EXPERIMENTAL SECTIONS. THE GRADES ARE APPROXIMATELY THE SAME ON BOTH SIDES OF THE TRACKS AND THOSE DESCENDING TOWARD THE NORTH ARE SHOWN AS NEGATIVE

116266—30——2

TABLE 1.—*Cost and description of experiments on Connecticut Avenue, Chevy Chase, Md.*

BITUMINOUS MACADAM (PENETRATION) EXPERIMENTS, SOUTH OF BRADLEY LANE, BUILT IN 1911

Experiment No.	Length	Area[1]	Original construction: Bituminous material or type	Gallons per square yard	Cost of surface, per square yard	Annual cost of surface treatments and maintenance in cents per square yard: 1912 Treatment	1912 Maintenance	1913 Treatment	1913 Maintenance	1914 Treatment	1914 Maintenance	1915 Treatment	1915 Maintenance	1916 Treatment	1916 Maintenance	1917 Treatment	1917 Maintenance
	Feet	*Square yards*			*Cents*												
1	513	1,581	Coal tar, refined	2.59	46.18					5.04	0.74		2.34		0.28		0.29
2	373	705	Coal tar, refined (modified)	4.15	64.42								2.29		.64		1.49
3	823	1,555	Fluxed native asphalt	1.96	64.69								.95		.38		.11
4	823	1,555	Oil asphalt (Gilsonite)	2.19	57.18				0.26		.16		2.58		.67		.78
5	819	1,447	Oil asphalt	2.25	58.27		17.98		.15		.06		2.91		.53		.70
6	876	1,555	Oil asphalt	2.09	68.22				.22				2.07		.57		.18
7A	449	849	Oil asphalt	1.86	54.80	8.01					.15		2.89		.29		.45
7B	69	130	Oil asphalt	2.26	59.96		1.09				.34		3.59				
			SURFACE TREATMENT EXPERIMENTS ON WATERBOUND MACADAM, SOUTH OF BRADLEY LANE, BUILT IN 1911														
8	782	1,477	Water-gas tar preparation	0.54	[2] 39.59			5.46					3.40	6.55	1.37	5.59	0.37
9	403	761	Asphaltic petroleum	.53	[2] 42.01			5.21		4.71	1.18	7.50	.88	8.76	1.48	5.93	.71
10	482	1,013	Residual petroleum	.79	[2] 44.31				1.88	8.11	20.17	7.80	5.24	8.90	4.08	5.36	1.39
11	201	377	Native asphalt emulsion	3.32	[2] 81.51										1.56		1.35
			EXPERIMENTS NORTH OF BRADLEY LANE, BUILT IN 1912														
1	635	1,498	Bituminous concrete (Topeka specification) 2 inches thick on 6-inch 1 : 3 : 7 cement concrete		186.62								0.22				
2	630	1,400	Bituminous concrete (District of Columbia specifications) 2 inches thick, on 6-inch 1 : 3 : 7 cement concrete and seal coat of 0.51 gallon fluxed native asphalt		195.65						0.29		.26		0.25		
3	1,840	4,178	Cement and oil-cement concrete as in experiment No. 2, surface treated with various types of bituminous materials		154.95						.03		.06	7.81	.29		0.04
4	771	1,744	Oil-cement concrete, 1 : 1¾ : 3 and 5 pints residual petroleum per bag of cement		150.25						.12		1.05		1.02		.15
5	1,339	3,013	Cement concrete, 1 : 1¾ : 3		142.29						.07		.68		.28		
6	980	2,055	Vitrified brick, with base as in experiments Nos. 1 and 2, 2-inch sand cushion grouted with 1 : 1 sand-cement		258.21						.06		.02		.08		.06

[1] Some of the sections were of varying widths.
[2] Includes cost of wearing course.

TABLE 2.—*Analyses of bituminous materials used in original construction of the experiments south of Bradley Lane*

Experiment number	1 and 2	2	3	4	5	6	7	8	9	10	10	11
Material	Coal tar, refined	Coal tar refined (light)	Fluxed native asphalt	Gilsonite oil asphalt	Oil asphalt	Oil asphalt	Oil asphalt	Refined water-gas tar preparation	Asphaltic petroleum [1]	Residual petroleum [2] 1912	Asphaltic petroleum 1914	Native asphaltic emulsion
Specific gravity 25°/25° C	1.258	1.219	1.058	0.974	0.999	0.989	0.973	1.113	0.949	0.976	0.964	1.038
Specific viscosity, Engler:												
1. 100° C., 100 c. c										13.1		
2. 50° C., 50 c. c								15.5				
3. 25° C., 50 c. c									118		113.3	
Float test:												
1. 50° C., seconds	150											
2. 32° C., seconds		47								205		
Flash point, °C									37		40	
Burning point, °C									68		85	
Melting point, °C			46	52	74	47	90					
Penetration, 25° C., 100 g., 5 seconds			128	146	73	94	55					
Per cent loss, 163° C., 5 hours, 20 g			2.14	.87	.05	.68	.44		27.10	.13	26.46	[3] 3.35
Per cent loss, 105° C., 5 hours, 20 g									16.71			52.54
Penetration on residue			[4] 56	[5] 99	[6] 65	[4] 79	[7] 50					[8] 195
Float test on residue, 50° C., seconds									[9] 100		81	[8] 72
Float test on residue, 32° C., seconds									[8] 32	229	226	
Percentage soluble in CS_2			94.51	99.81	99.46	99.59	99.82		99.88	99.74	99.92	36.30
Percentage organic insoluble			1.00	.12	.50	.28	.15		.08	.23	.06	1.10
Percentage inorganic insoluble			4.49	.07	.04	.13	.03		.04	.03	.02	3.30
Bitumen insoluble in 86° B. naphtha			20.78	21.13	24.68	20.10	26.20		9.10	9.93	7.43	
Fixed carbon, per cent			10.64	7.79	13.53	8.36	10.15		4.91	7.67	5.04	
Free carbon, per cent	29.60	23.35						3.04				
									Water			49.50
									Ammonia			.35
									Fatty and resin acids			9.45
Distillation, percentage by weight:												
Water	0	1.0						0				
Up to 110° C	[10] .2	[11] .7						0				
110° to 170° C	[10] .6	[12] .5						[11] .6				
170° to 270° C	[13] 10.2	[14] 16.8						[11] 24.8				
270° to 315° C	[10] 7.5	[15] 6.5						[11] 9.7				
Residue	[16] 81.4	[16] 74.3						[17] 64.9				
Total	99.9	99.8						100.0				

[1] Fairly thin fluid with strong naphtha odor.
[2] Viscous, sticky fluid.
[3] Loss in addition to loss at 105° C.
[4] Sticky, glossy surface.
[5] Mottled surface.
[6] Sticky, slightly mottled surface.
[7] Hard, fairly lustrous.
[8] Residue from percentage loss test at 105° C.
[9] Residue from percentage loss test at 163° C.
[10] Solid.
[11] Clear.
[12] Turbid.
[13] One-third solid.
[14] Two-thirds solid.
[15] One-sixth solid.
[16] Hard, dull, brittle.
[17] Sticky, semisolid. A 350° to 375° C. fraction showed 7.5 per cent insoluble in dimethyl sulphate.

TABLE 1.—*Cost and description of experiments on Connecticut Avenue, Chevy Chase, Md.*—Continued

BITUMINOUS MACADAM (PENETRATION) EXPERIMENTS, SOUTH OF BRADLEY LANE, BUILT IN 1911

Experiment No.	Annual cost of surface treatments and maintenance in cents per square yard—Continued																								Total maintenance 1912 to 1929 inclusive
	1918		1919		1920		1921		1922		1923		1924		1925		1926		1927		1928		1929		
	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	*Cents per sq. yd.*
1	19.12	1.47						0.04					9.68	1.37				1.90		0.72		3.13		0.26	46.38
2	22.42	2.95						.10						2.59						.89		11,51		2.52	47.40
3	20.84	1.40						.04						1.37				3.39		.65		1.04		1.25	31.42
4	19.40	1.63						.82						.99		8.96		3.51		3.29		13.02		0.00	56.07
5	16.61	2.75		0.32				.05						3.38		6.33		4.82				5.67		18.54	80.80
6	17.54	1.27						.04				1.51		3.77		1.34		2.37				5.77		1.08	37.73
7	20.93	1.98						.07				1.32		6.22		4.05		8.84		3.36		3.97		3.03	64.76
	SURFACE TREATMENT EXPERIMENTS ON WATERBOUND MACADAM, SOUTH OF BRADLEY LANE, BUILT IN 1911																								
8		1.31	11.96	0.57				0.05				6.21	9.55	3.61		5.79	11.41	11.94		0.33		4.39		9.56	99.42
9	11.56	3.43	11.36	1.08	8.18		17.21	4.23				7.99		20.76		22.47		18.47		.65		25.15		30.39	219.28
10	9.11	2.04	10.49	.81	6.89	2.27	17.54	1.06				2.95		8.85		6.74		21.36		2.33		16.18		18.25	189.80
11	12.80	6.89			14.99	4.75		6.44				2.65	7.91	2.75		6.63		9.18		1.32		8.91		0.00	88.13
	EXPERIMENTS NORTH OF BRADLEY LANE, BUILT IN 1912																								
1				0.29				0.61		2.67		1.00		1.68		0.56		1.33		1.18		6.71		4.90	21.15
2				.30				.65		2.47		.79		1.93		1.06		.39		1.86		1.58		0.78	12.61
3		0.07		.20		0.39		.16	10.45	1.38		3.52		1.54		1.74		7.58		2.53		5.60		5.14	48.53
4		.29		.47		.94		1.93	10.46	3.31		4.23		4.99		1.68		11.90		4.86		3.26		4.90	55.56
5		.47		.27		.55		.22	10.45	1.69		3.49		1.55		.42		1.53		1.07		3.99		3.59	30.32
6		.09		.66		.66		.03		.66		.55				.28		4.42		1.69		[3] 1.28		[3] 2.11	12.65

[3] Cost of maintaining sections not affected by fill settlement.

TABLE 3.—*Character and extent of experimental sections constructed on Connecticut Avenue north of Bradley Lane*

Experiment No.	Experiment Section	Location From—	Location To—	Area	Type	Aggregate
				Square yards		
1	A	0−15	3+19	1,498	2 inches bituminous concrete (Topeka specification).	Limestone.[1]
	B	3+19	6+20		....do	Trap.[1]
2	A	6+20	9+04	1,400	2 inches bituminous concrete (District of Columbia specification).	Limestone.[1]
	B	9+04	12+50		....do	Trap.[1]
3	A	12+50	15+84	4.178	Cement concrete surface treated with bituminous material.	Gravel.
	B	15+84	21+60		....do	Limestone.
	C	21+60	25+03		....do	Gravel.
	A	25+03	27+29		Oil-cement concrete, surface treated with bituminous material.	Limestone.
	B	27+29	30+90		....do	Gravel.
4	C	30+90	33+60	1,744	Oil-cement concrete	Do.
	D	42+50	44+61		....do	Limestone.
	E	46+10	49+00		....do	Trap.
5	A	33+60	37+85	3,013	Cement concrete	Gravel.
	B	37+85	42+50		....do	Limestone.
	C	44+61	46+10		....do	Do.*
	D	49+00	52+00		....do	Trap.
6	......	52+00	61+80	2,055	Vitrified brick	

[1] Aggregate used in the bituminous concrete. Gravel used in the cement concrete of the base.

omitted from this report as no retreatments have been applied since 1926.

TABLE 4.—*Analyses of bituminous concrete mixtures*

Coarse aggregate	Experiment No. 1 — Topeka specification		Experiment No. 2 — District of Columbia specification	
	Limestone	Trap	Limestone	Trap
Bitumen soluble in CS_2	7.1	8.7	6.7	6.7
Sieve analysis of aggregate:				
Pass 1¼-inch screen, retained 1-inch screen			2.1	0
Pass 1-inch screen, retained ¾-inch screen			13.2	6.5
Pass ¾-inch screen, retained ½-inch screen	1.8	2.0	16.7	15.2
Pass ½-inch screen, retained ¼-inch screen	11.3	14.0	13.5	19.2
Pass ¼-inch screen, retained ⅛-inch screen	20.5	13.0	12.7	13.5
Pass ⅛-inch screen, retained 10-mesh sieve	14.5	8.8	6.5	7.4
Pass 10-mesh sieve, retained 20-mesh sieve	15.8	17.0	7.6	8.7
Pass 20-mesh sieve, retained 30-mesh sieve	5.8	7.3	3.5	4.0
Pass 30-mesh sieve, retained 40-mesh sieve	3.8	4.0	3.3	3.3
Pass 40-mesh sieve, retained 50-mesh sieve	1.6	2.3	2.0	1.9
Pass 50-mesh sieve, retained 80-mesh sieve	3.3	4.4	4.1	4.0
Pass 80-mesh sieve, retained 100-mesh sieve	1.5	1.8	1.1	1.2
Pass 100-mesh sieve, retained 200-mesh sieve	2.9	4.6	2.0	2.6
Pass 200-mesh sieve	10.1	12.1	5.0	5.8
Total	100.0	100.0	100.0	100.0

SECTIONS 1 TO 8 OF BITUMINOUS MACADAM CONTINUE TO GIVE GOOD SERVICE

Section 1, although still maintained by the bureau, has lost considerable of its value as an experiment due to the reconstruction work done by the municipal forces around Chevy Chase Circle in November and December

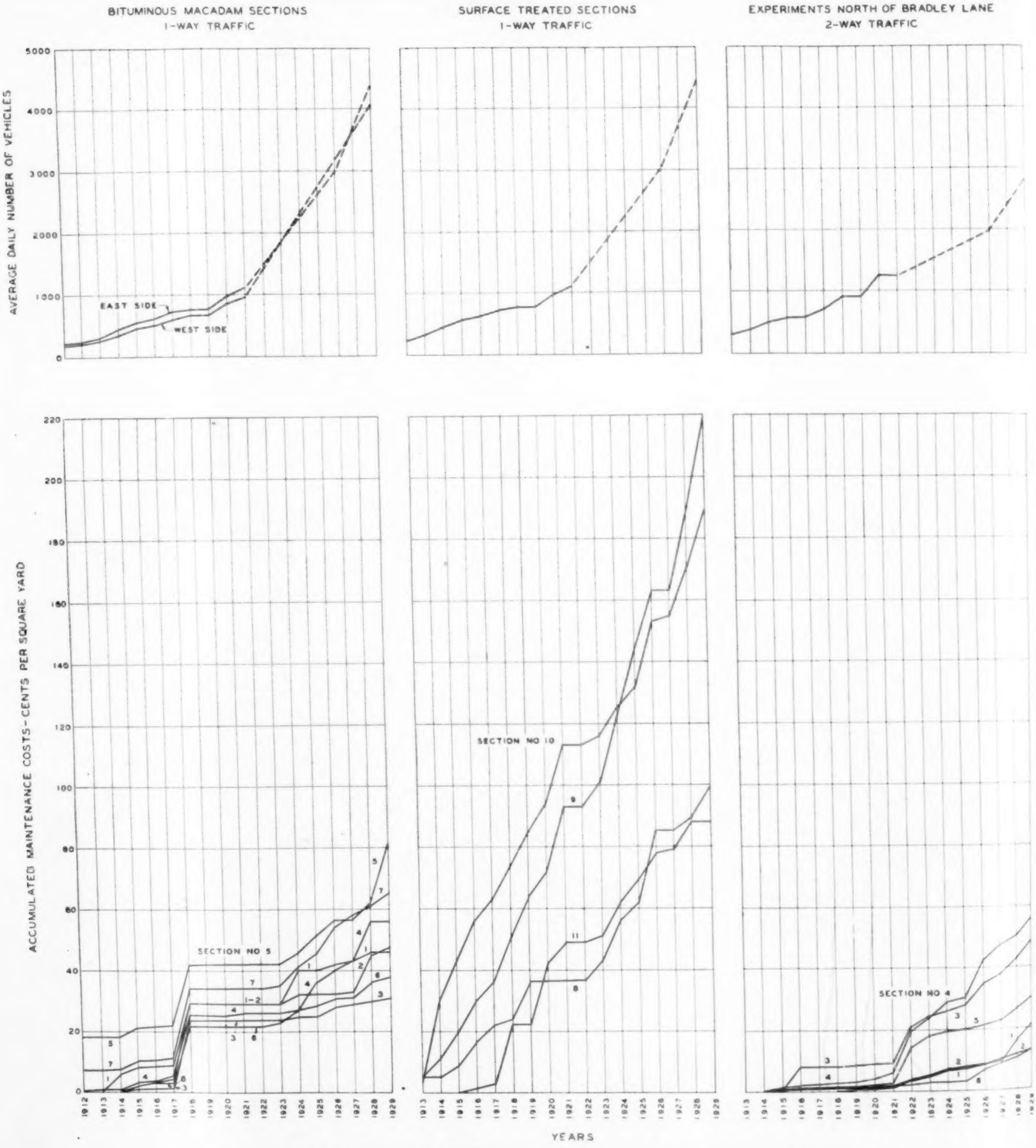


FIGURE 2.—MAINTENANCE COSTS AND TRAFFIC ON EXPERIMENTAL SECTIONS

1927. This work was incidental to relaying the curb and gutter and involved the rebuilding of a large portion of the section.

As shown in Table 1, the maintenance cost during 1928 for sections 1, 3, 5, 6, and 7 was about normal while that of sections 2 and 4 was somewhat higher than during former years. The maintenance on the latter two sections has consisted largely of repairing depressions which developed from foundation settlement, particularly along the west gutter. As stated in the previous report, the drainage afforded the sections south of Bradley Lane at the best was only fair. The streetcar tracks offered a means for water to enter the foundations and the cobble gutters did not long remain satisfactory as drainage structures. The relatively high maintenance cost of section 5 in 1929 was also due to surface settlement. This section, in its early life, required considerable patching because of foundation

TABLE 5.—*Analyses of bituminous materials used in original construction of experiments north of Bradley Lane*

	Experiments Nos. 1 and 2: Bituminous concrete—fluxed native asphalt	Experiment No. 3 (surface treatments)							Experiments Nos. 3 and 4: Oil-cement concrete—residual petroleum
		Sections A and G: Refined coal tar	Sections D and F: Water-gas tar preparation	Sections B and H: Water-gas tar preparation	Section E: Fluxed native asphalt	Sections C, D and I: Fluxed native asphalt	Section F: Oil asphalt	Section J: Oil asphalt	
Specific gravity, 25°/25° C	1.074	1.219	1.108	1.144	1.045	1.043	1.031	1.012	0.933
Specific viscosity, Engler:									
1. 50°/50° C., 100 c. c									27.8
2. 30°/30° C., 50 c. c			14.0						
Penetration, 25° C., 100 g., 5 seconds	60				184	148	[1] 26	147	
Float test, 32° C., seconds		83		207					
Float test, 50° C., seconds		40		75					
Melting point, °C	53				39	48	92	52	
Loss 163° C., 5 hours, 20 g., per cent	2.86				3.50	2.73	.32	.59	2.63
Penetration on residue	29				63	64	18	103	
Float test on residue at 32° C., seconds									95
Per cent soluble in CS_2	93.56				94.70	96.56	99.74	99.72	99.90
Per cent insoluble in CS_2	1.86				1.23	1.35	.17	.17	.08
Per cent inorganic insoluble	4.58				4.07	2.09	.09	.11	.02
Per cent bitumen insoluble in 86° B., naphtha	23.83				19.60	21.87	34.84	21.72	2.31
Per cent fixed carbon	11.20				9.83	11.17	15.62	10.92	3.01
Per cent free carbon		16.29	.25	.95					
Distillation, per cent by weight:									
Water		0	0	0					
Up to 110° C		[2] .5	[3] 1.4	[3] .1					
110° to 170° C		[4] .2	[3] .4	[3] .2					
170° to 270° C		[5] 14.7	[3] 16.3	[3] 6.4					
270° to 315° C		[6] 8.7	[7] 17.0	[8] 10.4					
Residue		75.8	64.8	82.8					
Total		99.9	99.9	99.9					

[1] Penetration at 0° C. (200 g., 1 minute) 14; penetration at 46° C. (50 g., 5 seconds) 58.
[2] One-half solid.
[3] Clear.
[4] Cloudy.
[5] Two-thirds solid.
[6] Solid.
[7] Clear. Showed 7.5 per cent insoluble in dimethyl suplhate. A 315° to 350° C. fraction showed 7.5 per cent, and a 350° to 375° C. fraction showed 17.5 per cent insoluble in dimethyl sulphate.
[8] Clear. This fraction and also a 315° to 350° C. fraction and a 350° to 375° C. fraction each showed 7.5 per cent insoluble in dimethyl sulphate.

TABLE 6.—*Mechanical analyses of coarse aggregates used in concrete Experiments 3, 4, and 5, north of Bradley Lane*

Size	Gravel	Limestone	Trap
Pass 2½-inch, retained on 1½-inch screen	2.3		
Pass, 1½-inch, retained on 1¼-inch screen	10.8		
Pass 1¼-inch, retained on 1-inch screen	24.0	13.2	6.6
Pass 1-inch, retained on ¾-inch screen	25.8		19.5
Pass ¾-inch, retained on ½-inch screen	26.4	34.2	40.6
Pass ½-inch, retained on ¼-inch screen	9.8	47.4	24.9
Pass ¼-inch	.9	5.2	8.4
Total	100.0	100.0	100.0

weakness, and as the weak areas were eliminated the maintenance cost decreased. A renewal of this type of failure such as developed during the past year will undoubtedly affect the future behavior of this section, particularly in view of the larger volume of traffic now carried.

At the present time all of the sections are in good condition. The surfaces are somewhat wavy in spots but are intact and free from raveling. Except for possible failure due to foundation conditions, they should continue to give satisfactory service for some time at moderate cost.

TABLE 7.—*Tests on vitrified brick used in experiment No. 6 north of Bradley Lane*

[Length of section, 978.1 feet]

Section	Length	Type of brick	Rattler loss	Water absorption	Description
	Feet		*Per cent*	*Per cent*	
A	51.5	Shale, wire cut lug	21.12	1.39	Hard-burned brick having a good structure.
B	67.5	do	16.36	1.31	Medium hard-burned brick having a very good structure.
C	108.7	Shale, re-pressed	25.57	.88	Brick well vitrified; losses in rattler mainly due to chipping.
D	105.0	do	17.67	1.65	Brick molded from coarsely ground shale; had a fairly good structure and was hard burned.
E	111.4	do	22.04	1.10	Brick very hard burned; losses in rattler due to chipping.
F	69.4	do	18.80	1.81	Brick molded from coarsely ground clay; had a good structure.
G	60.5	do	27.92	2.29	Medium hard burned brick which wear evenly though excessively in the rattler test.
H	67.9	do	22.68	3.74	Medium hard burned brick made from finely ground clay and having a fairly good structure.
I	50.0	do	22.59	2.86	Medium hard brick made from coarsely ground clay and wearing down uniformly in the rattler.
J	61.3	Fire clay, re-pressed	19.11	1.56	Brick made from coarsely ground fire clay; had an excellent structure, free from laminations; not burned very hard.
K	54.7	do	37.68	2.38	Comparatively soft-burned brick made from coarsely ground fire clay; wear in rattler excessive though uniform.
L	58.8	Shale, re-pressed	38.89	4.04	Comparatively soft-burned brick made from coarsely ground clay; wear in rattler excessive though uniform.
M	60.1	Fire clay, re-pressed	24.31	3.73	Fairly soft-burned brick made from medium finely ground clay; worn down evenly by rattler.
N	51.3	Fire clay, wire-cut lug	31.19	3.68	Losses in rattler due mainly to open laminations; brick burned hard.

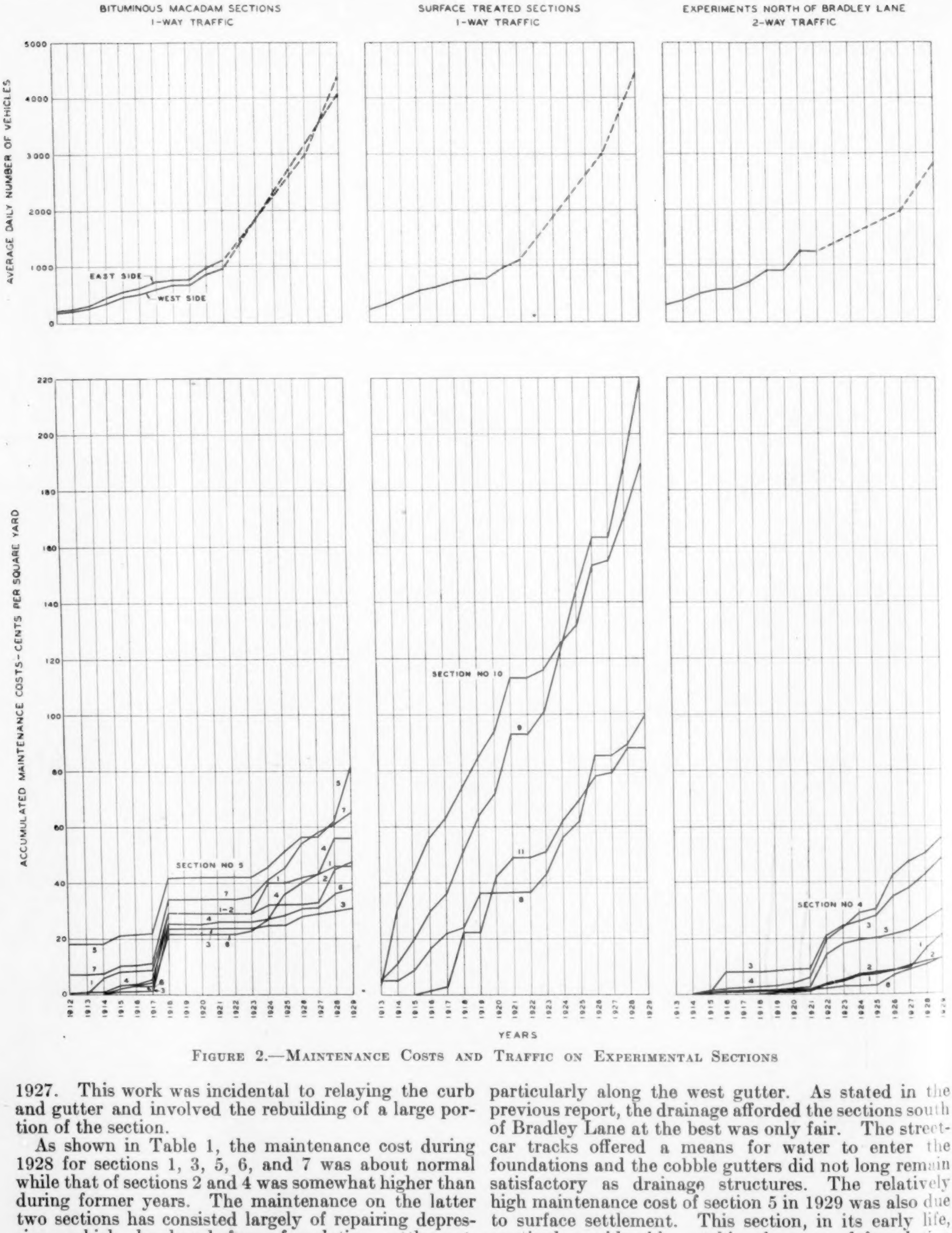


FIGURE 2.—MAINTENANCE COSTS AND TRAFFIC ON EXPERIMENTAL SECTIONS

1927. This work was incidental to relaying the curb and gutter and involved the rebuilding of a large portion of the section.

As shown in Table 1, the maintenance cost during 1928 for sections 1, 3, 5, 6, and 7 was about normal while that of sections 2 and 4 was somewhat higher than during former years. The maintenance on the latter two sections has consisted largely of repairing depressions which developed from foundation settlement, particularly along the west gutter. As stated in the previous report, the drainage afforded the sections south of Bradley Lane at the best was only fair. The street-car tracks offered a means for water to enter the foundations and the cobble gutters did not long remain satisfactory as drainage structures. The relatively high maintenance cost of section 5 in 1929 was also due to surface settlement. This section, in its early life, required considerable patching because of foundation

Table 5.—*Analyses of bituminous materials used in original construction of experiments north of Bradley Lane*

	Experiments Nos. 1 and 2: Bituminous concrete—fluxed native asphalt	Experiment No. 3 (surface treatments): Sections A and G: Refined coal tar	Experiment No. 3: Sections D and F: Water-gas tar preparation	Experiment No. 3: Sections B and H: Water-gas tar preparation	Experiment No. 3: Section E: Fluxed native asphalt	Experiment No. 3: Sections C, D and I: Fluxed native asphalt	Experiment No. 3: Section F: Oil asphalt	Experiment No. 3: Section J: Oil asphalt	Experiments Nos. 3 and 4: Oil-cement concrete—residual petroleum
Specific gravity, 25°/25° C	1.074	1.219	1.108	1.144	1.045	1.043	1.031	1.012	0.933
Specific viscosity, Engler:									
1. 50°/50° C., 100 c. c									27.8
2. 30°/30° C., 50 c. c			14.0						
Penetration, 25° C., 100 g., 5 seconds	60				184	148	[1] 26	147	
Float test, 32° C., seconds		83		207					
Float test, 50° C., seconds		40		75					
Melting point, °C	53				39	48	92	52	
Loss 163° C., 5 hours, 20 g., per cent	2.86				3.50	2.73	.32	.59	2.63
Penetration on residue	29				63	64	18	103	
Float test on residue at 32 ° C., seconds									95
Per cent soluble in CS_2	93.56				94.70	96.56	99.74	99.72	99.90
Per cent insoluble in CS_2	1.86				1.23	1.35	.17	.17	.08
Per cent inorganic insoluble	4.58				4.07	2.09	.09	.11	.02
Per cent bitumen insoluble in 86° B., naphtha	23.83				19.60	21.87	34.84	21.72	2.31
Per cent fixed carbon	11.20				9.83	11.17	15.62	10.92	3.01
Per cent free carbon		16.29	.25	.95					
Distillation, per cent by weight:									
Water		0	0	0					
Up to 110° C		[2] .5	[3] 1.4	[3] .1					
110° to 170° C		[4] .2	[3] .4	[3] .2					
170° to 270° C		[5] 14.7	[3] 16.3	[3] 6.4					
270° to 315° C		[6] 8.7	[7] 17.0	[8] 10.4					
Residue		75.8	64.8	82.8					
Total		99.9	99.9	99.9					

[1] Penetration at 0° C. (200 g., 1 minute) 14; penetration at 46° C. (50 g., 5 seconds) 58.
[2] One-half solid.
[3] Clear.
[4] Cloudy.
[5] Two-thirds solid.
[6] Solid.
[7] Clear. Showed 7.5 per cent insoluble in dimethyl suplhate. A 315° to 350° C. fraction showed 7.5 per cent, and a 350° to 375° C. fraction showed 17.5 per cent insoluble in dimethyl sulphate.
[8] Clear. This fraction and also a 315° to 350° C. fraction and a 350° to 375° C. fraction each showed 7.5 per cent insoluble in dimethyl sulphate.

Table 6.—*Mechanical analyses of coarse aggregates used in concrete Experiments 3, 4, and 5, north of Bradley Lane*

Size	Gravel	Limestone	Trap
Pass 2½-inch, retained on 1½-inch screen	2.3		
Pass, 1½-inch, retained on 1¼-inch screen	10.8		
Pass 1¼-inch, retained on 1-inch screen	24.0	13.2	6.6
Pass 1-inch, retained on ¾-inch screen	25.8		19.5
Pass ¾-inch, retained on ½-inch screen	26.4	34.2	40.6
Pass ½-inch, retained on ¼-inch screen	9.8	47.4	24.9
Pass ¼-inch	.9	5.2	8.4
Total	100.0	100.0	100.0

weakness, and as the weak areas were eliminated the maintenance cost decreased. A renewal of this type of failure such as developed during the past year will undoubtedly affect the future behavior of this section, particularly in view of the larger volume of traffic now carried.

At the present time all of the sections are in good condition. The surfaces are somewhat wavy in spots but are intact and free from raveling. Except for possible failure due to foundation conditions, they should continue to give satisfactory service for some time at moderate cost.

Table 7.—*Tests on vitrified brick used in experiment No. 6 north of Bradley Lane*

[Length of section, 978.1 feet]

Section	Length	Type of brick	Rattler loss	Water absorption	Description
	Feet		*Per cent*	*Per cent*	
A	51.5	Shale, wire cut lug	21.12	1.39	Hard-burned brick having a good structure.
B	67.5	do	16.36	1.31	Medium hard-burned brick having a very good structure.
C	108.7	Shale, re-pressed	25.57	.88	Brick well vitrified; losses in rattler mainly due to chipping.
D	105.0	do	17.67	1.65	Brick molded from coarsely ground shale; had a fairly good structure and was hard burned.
E	111.4	do	22.04	1.10	Brick very hard burned; losses in rattler due to chipping.
F	69.4	do	18.80	1.81	Brick molded from coarsely ground clay; had a good structure.
G	60.5	do	27.92	2.29	Medium hard burned brick which wear evenly though excessively in the rattler test.
H	67.9	do	22.68	3.74	Medium hard burned brick made from finely ground clay and having a fairly good structure.
I	50.0	do	22.59	2.86	Medium hard brick made from coarsely ground clay and wearing down uniformly in the rattler.
J	61.3	Fire clay, re-pressed	19.11	1.56	Brick made from coarsely ground fire clay; had an excellent structure, free from laminations; not burned very hard.
K	54.7	do	37.68	2.38	Comparatively soft-burned brick made from coarsely ground fire clay; wear in rattler excessive though uniform.
L	58.8	Shale, re-pressed	38.89	4.04	Comparatively soft-burned brick made from coarsely ground clay; wear in rattler excessive though uniform.
M	60.1	Fire clay, re-pressed	24.31	3.73	Fairly soft-burned brick made from medium finely ground clay; worn down evenly by rattler.
N	51.3	Fire clay, wire-cut lug	31.19	3.68	Losses in rattler due mainly to open laminations; brick burned hard.

RESULTS ON SURFACE-TREATED SECTIONS INDICATE DESIRABILITY OF USE OF PRIMING COAT IN CONSTRUCTION

The work previously referred to, which was done around Chevy Chase Circle, likewise affected section 11 of this group.

During 1928 sections 8, 9, and 10 required fairly heavy maintenance. On sections 9 and 10 the surface mat slipped upon the stone base and became very wavy This displacement was intensified during the winter months when chains were used on cars. Where the base became exposed raveling developed. Defective areas were repaired by removing the bituminous mat to the base stone, which was then painted with tar and a patch made with a mixture of tar and stone. About 20 per cent of the areas of sections 9 and 10 were repaired in this manner.

mat unless the surface so treated is well bonded and of a character to which the treatment will adhere. It was observed that the surface treatments applied to a penetration macadam adhered satisfactorily and that a wear-resisting stable mat could be successfully built up. The same materials, however, applied directly to the unprimed, water-bound macadam proved unsuccessful as there was no bond between the treatment and the foundation.

Sections 9 and 10 have been repeatedly re-treated and have developed mats which are not well bonded to the base and which are consequently subject to displacement. On the other hand, section 8, on which a light water-gas tar was used, has always remained stable. The retreatments required during its early life were due to the fact that the light tar used would

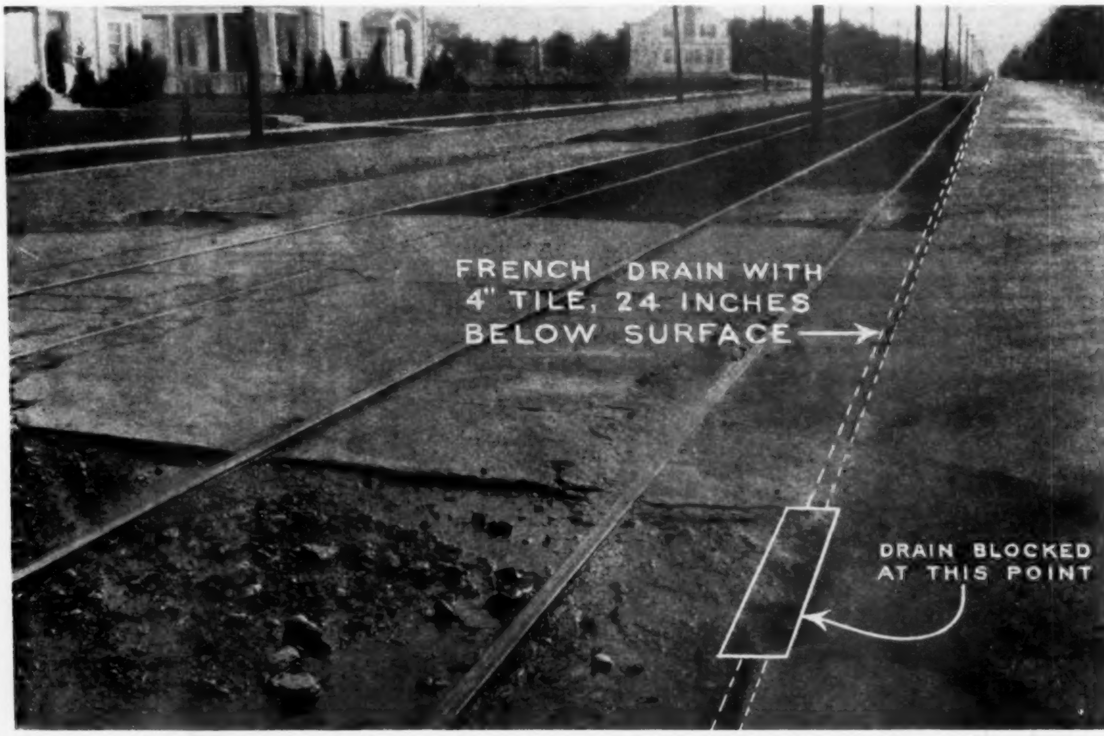


FIGURE 3.—SECTION 3, NORTH OF BRADLEY LANE, FEBRUARY, 1929. HEAVY FLOW OF WATER RISING TO THE SURFACE IMMEDIATELY UNDER WEST RAIL ON NORTH SIDE OF VIRGILIA STREET INTERSECTION. NOTE BITUMINOUS PATCH ON THE RIGHT WHICH WAS NECESSITATED BY SETTLEMENT CAUSED BY THE IMPROPER BACK FILLING OF A SERVICE CUT. THE EAST DRAIN WAS STOPPED AT THIS POINT.

Maintenance for 1929 was normal except for sections 8 and 9. Section 8 because of its dry-appearing surface was practically covered with a light patch and at present is apparently in very good condition. Approximately two-thirds of the surface mat on section 9 had to be replaced. Except for a strip about 2 feet in width along the car tracks and one 3 feet wide next to the gutter, the surface is now uniform in appearance and the entire section is fairly smooth.

The behavior of the surface-treated sections during the past two years emphasizes still further the advantage of using a suitable priming material before applying a surface treatment. In a study of the Bradley Lane [2] experiments and those on the Department of Agriculture grounds [3] it was shown that it is practically impossible to retain stability in a bituminous surface

not hold the stone cover. This light tar penetrated readily and bonded the surface of the base but in so doing left no binder to hold the stone. For such a condition, subsequent re-treatments can be successfully used to build up a wear-resisting mat but, in the case of unprimed sections such as Nos. 9 and 10, additional treatments serve only to build up a thicker mat and add little to the durability of the structure as a whole. The surface mat merely rests upon an unbonded base which itself is subject to movement or displacement.

The cost of maintaining the surface-treated sections since the time of construction has been over three times that of the bituminous macadams, being 9.15 cents and 2.86 cents per square yard, respectively. Experiment No. 8 has remained the most economical of the surface-treated group but its cost of maintenance has been more than double that of the average of the bitu-

[2] Reported in PUBLIC ROADS, February, 1929.
[3] PUBLIC ROADS, October, 1929.

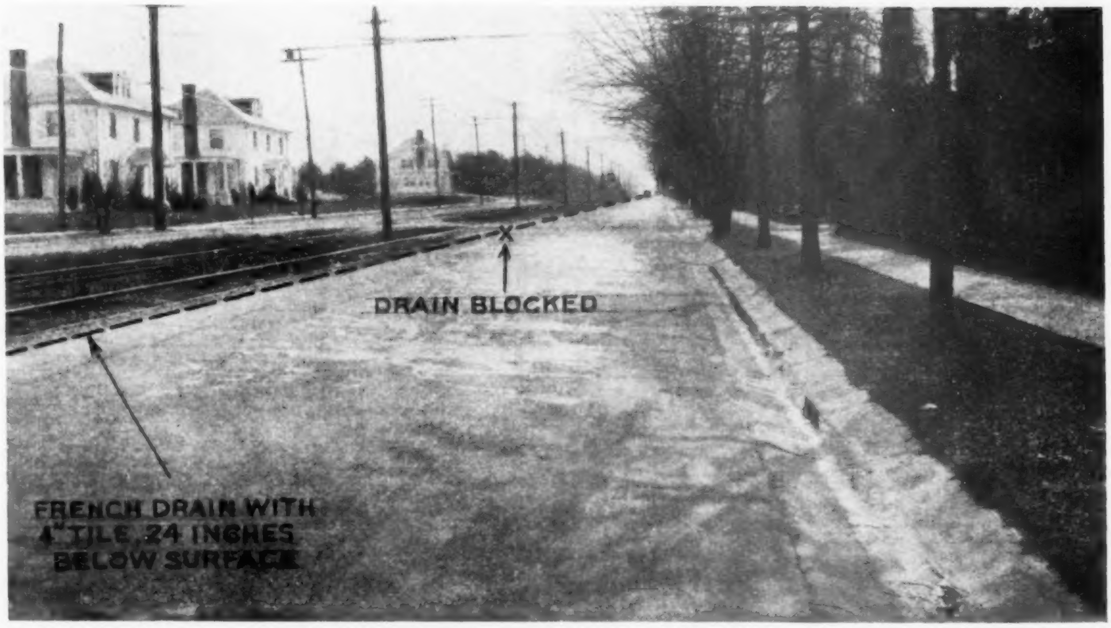


FIGURE 4.—SECTION 3, NORTH OF BRADLEY LANE, FEBRUARY, 1929. APPEARANCE OF SECTION JUST NORTH OF VIRGILIA STREET. SHINY AREAS ARE WATER WHICH HAS COME UP THROUGH THE CRACKS IN THE CONCRETE PAVEMENT. NOTE WATER WHICH HAS FLOWED FROM THE SURFACE INTO THE GUTTER AT THE RIGHT

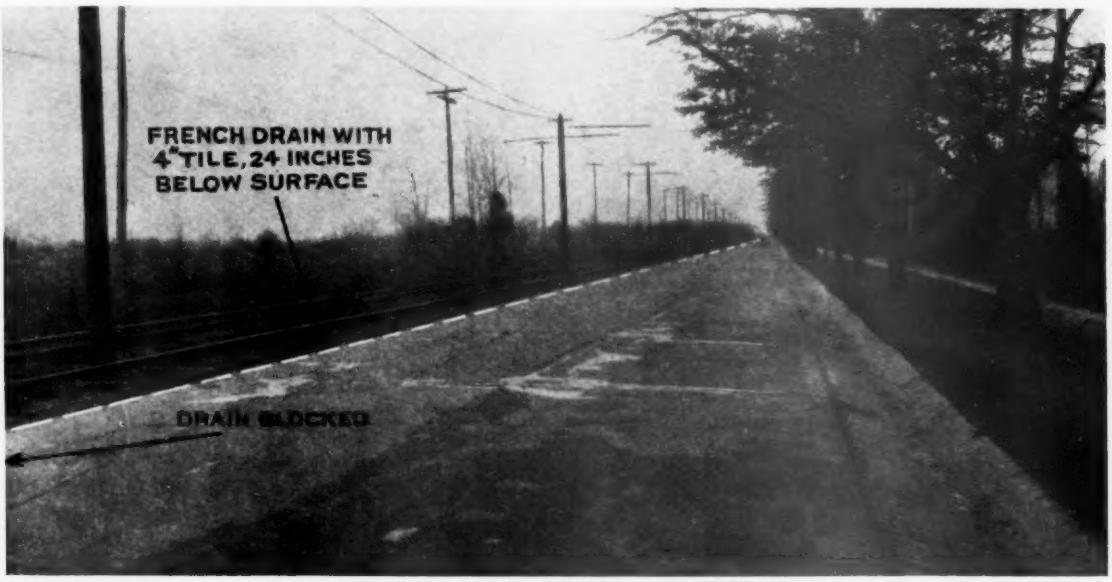


FIGURE 5.—VIEW TAKEN JUST NORTH OF CYPRESS STREET AND A FEW FEET SOUTH OF THE ENTRANCE TO COLUMBIA COUNTRY CLUB. LIGHT AREAS ARE POOLS OF WATER COLLECTING ALONG CRACKS IN THE CONCRETE. DRAIN LINE WAS BLOCKED AS INDICATED

minous macadam sections. In this comparison of costs, section 11, which is more closely allied to penetration construction, has been omitted.

FAILURE OF DRAINAGE SYSTEM AFFECTS RESULTS ON SEVERAL SECTIONS

The six experimental sections north of Bradley Lane built in 1913 differ from the macadams and surface treated sections in that they are higher type surfaces laid upon a concrete base. Their construction and history have also been described in PUBLIC ROADS of May, 1928.

As noted in that report, the subgrade upon which the six sections are located appeared such at the time of construction that French drains were deemed necessary on both edges of the pavement.

Maintenance during 1928 and 1929 consisted of routine repairs, practically all of which were caused by foundation failures.

An investigation of the drainage system made in February, 1929, by the bureau showed that it had practically ceased to function. Immediately following a heavy rain in February it was noted that a considerable volume of water was flowing to the surface

of the concrete sections through the cracks. This condition was found on all the concrete sections but was more pronounced from Woodbine Street, south, which area embraces the whole of Experiment 3. It was also noted that the sewers were not receiving any flow from either side drain or through the cross drains. Upon digging down into the French drains it was found that wherever one of the utility companies had made a cut through the drain line they had failed to replace either the tile or stone but had back-filled their own trench with earth and thereby effectively destroyed the drainage system. The number of tiles thus removed from the line varied from eight to as many as twenty.

Preceding the failure of the drainage system the east drain line apparently took care of the area occupied by the street-car tracks, but with its failure this unpaved area offered an additional opportunity for surface water to enter the foundation of the experiments. At the time of inspection, February, 1929, it was observed that no free water was in evidence at Virgilia Street south of the intersection in the car-track area, but immediately north of the intersection, in the west car track, a considerable stream of water was rising to the surface, some of which flowed over the rail and onto the concrete pavement. This condition is illustrated in Figure 3. At this intersection the east drain was plugged up and as the accumulated water could travel no farther through the French drain, it flowed to the surface. This observed action was made use of in locating other breaks in the drain lines. Water was forced into the line under pressure until a flow to the surface indicated an obstruction. When the drain lines were uncovered at these points they were invariably found to be obstructed. When the defects had been corrected there was an unbroken flow through the east drain to its outlet at the north end of Experiment 6, and also on the west side of the sections which emptied into the sewer manholes.

The number of breaks in the tile line which were found and repaired was as follows: Experiment 1, 2 breaks; Experiment 2, 2 breaks; Experiment 3, 6 breaks; Experiment 4, 1 break.

The time at which the drainage system failed to function or, if it gradually failed, just what share of the maintenance required on the sections, or apparent slab failures were due to this fact, can not be estimated. The records of the sanitary commission show that most of the service connections were made in 1922, and it is interesting to note that the cost of maintaining the concrete sections has been materially higher since that date. The maintenance records show an average annual cost prior to 1922 of only 0.29 cent per square yard, but for the period from 1922 to 1930 it has averaged 2.86 cents per square yard. It was shown in the previous report that there was little variation in the compressive strength of the concrete cores taken, regardless of the size of the slab from which they were taken, and observations made revealed no evidence of structurally unsound concrete other than the numerous cracks which resulted in many small slabs. This excessive cracking and the corresponding increase in maintenance cost may have been influenced to some extent by the increased volume of traffic as shown in Figure 2, but it seems likely that they were influenced to a greater degree by foundation failures resulting from the causes described above.

FIGURE 6.—METHOD OF MAKING REPAIRS ON THE CONCRETE SECTIONS IN THE BADLY CRACKED AREAS

It was further stated in the previous report that "there was no appearance of settlement or subgrade failure, and for this reason those areas which seem to have reached their service limit as a concrete pavement might economically serve as a base for some type of surfacing." The reference to settlement or subgrade failure appeared true at that time, but, in view of the more recent study, the statement needs correcting. It is hardly reasonable to expect that such a great amount of cracking would have occurred had the slab received proper support. Just prior to February, 1929, some settlement of the concrete was noted, especially in the badly cracked areas. The amount was not great, but it did require the placing of many small patches to retain a reasonably smooth riding surface. After the drains were repaired and again began to function no slab settlement was noted and maintenance then consisted of filling cracks. There has been no recurrence of water coming to the surface, and, in general, the sections

(Continued on page 80)

SOME POINTS OF CONTACT BETWEEN SOIL SCIENCE AND HIGHWAY ENGINEERING[1]

By J. S. JOFFE, New Jersey Agricultural Experiment Station

THE principles underlying the elucidation of the natural sciences such as zoology and botany are applicable also to soil science. The soil is a natural body just as a tree or an animal. It is to be looked upon as a distinct organism with definite morphological and physiological features, with specific properties of physical build, chemical composition, and biological make-up. Soil science is concerned with the soil body as found in nature; its anatomy and physiology and its behavior toward the forces which are responsible for its creation. The first person to bring out the natural body features of the soil in relation to the forces in nature responsible for the creation of definite body types was a Russian scientist, Dokuchaev. The Russian school of soil science is known, therefore, as the Dokuchaev school.

Like any other of the natural sciences, soil science began with the descriptive phase. The soil body was dissected, cut open vertically, and the exposed anatomy noted and described. In a natural state it revealed a definite construction or build, consisting of distinct layers, known as horizons, which are specific in their morphological characters, irrespective of the geographic position of the soil or the underlying geologic formation provided it is located in identical climatic zones. The horizons exposed in a vertical cut of the soil body are genetically related and as a unit they represent what is known as the soil profile. Therefore a soil is a natural body consisting of definite layers or horizons made up of materials formed by a group of soil formers. Most of the materials that make up the soil body originate from the earth's mantle. The soil formers, which include the active factors such as the climate and biosphere, and the passive factors such as the parent material, micro-relief, age of land, and human activity, are responsible for the formation of the soil body.

It is to be understood that the parent material could be either native rock upon which the soil body has been formed or some material which might have been a part of a soil body before, as in the case of the so-called transported soils, or it might even be some geologic formation like peat, clay, marl, or sand. Soil material is therefore not to be identified with the soil as a natural body. A soil body ceases to be one as soon as its virgin make-up has been disturbed; it continues to be soil material from which a soil might form again in the course of time under the influence primarily of the active agents—the climate and the biosphere.

From what has been said it is clear that various parent materials will give rise to one and the same type of soil, provided all other conditions for the activities of the other soil formers are alike. And we do find the soil type known as chernozem on such variable parent material as loess, glacial deposits, marine and lake sands and clays, limestone, sandstone, and shale. On the other hand, on one parent material unlike soil types will develop, provided the conditions for the activities of the other soil formers are not alike. Thus granites in Georgia yield the typical yellow-red soil, whereas the same granites in southern California form a different kind of soil.

GEOGRAPHIC DISTRIBUTION OF SOIL TYPES DISCUSSED

Geographically, soils are distributed with a certain natural regularity in the same way as animals and plants. Just as any particular climatic belt is responsible for a definite flora and fauna, it is responsible also for a definite soil type or types. The habitus of the profile, its morphology, and chemical composition differ in each climatic belt. As a result, we have several zonal types of soil.

In Europe and Asia where the isohyetal lines are more or less parallel to the isothermal lines, i. e., as we move from the north southward the temperature increases, and the rainfall decreases. There the soil types are distributed parallel to the climatic belts; in the northern regions we find the tundra zone, a type of soil corresponding to this climatic zone: in the southern portion of the northern region and in part of the temperate region covered with conifers and deciduous forests, a type of soil knows as the podzol is distributed. South of this region—in the temperate region—the slightly podzolized type and forest steppe type of grayish brown soil is distributed, followed in the southern portion of the temperate region—where the somewhat semiarid regions are reached—with the chernozem soil. As one moves into the semiarid and arid regions, one finds the chestmut soils, and still farther south—in approaching the semidesert—one meets the gray soils. In the Tropics with a high temperature and high rainfall the well-known laterite type of soil is developed.

In the North American continent, especially in the eastern part, the direction of the isohyetal lines is, in general, perpendicular to the direction of the isothermal lines, and the geographic distribution of the zonal types does not follow the north and south direction as they do in Europe and Asia. It is these specific climatic features of the North American continent that necessitated a slightly different approach to the soils of the United States when studied from the standpoint of their profile development.

Dr. C. F. Marbut, the prominent American soil investigator, of the Bureau of Chemistry and Soils, United States Department of Agriculture, divided the soils of the United States into two large groups, (1) pedalfers,[2] and (2) pedocals.[2] The pedalfers are soils that tend to accumulate iron and aluminum and have no lime carbonate horizon accumulation, even if the soils have limestone as parent material. The accumulation of lime carbonate and other salts is a characteristic feature of the pedocals.

SOIL TYPES OF NEW JERSEY HAVE DEFINITE CHARACTERISTICS

For the present it will suffice to keep in mind these two broad groups. An analysis of the soil types found

[1] This paper is the summary of a lecture given before the Fifth Annual Short Course n Highway Engineering, Rutgers University. Journal Series paper, New Jersey Agricultural Experiment Station, Department of Soil Chemistry and Bacteriology.

[2] The prefix "ped" comes from the word "pedology"; "alfer" is apparently an abbreviation for aluminum and ferrum; the "cal" in pedocal comes from the word "calcium."

in New Jersey, which are located in the pedalfer group, will be in order. Those interested in the classification scheme in its entirety are referred to the paper by Doctor Marbut (*5*).

An examination of the virgin soils in New Jersey upon exposure of a profile cut will in general reveal the following: On the surface there is a dark-colored layer, 2 to 3 centimeters thick, of leaf mold consisting of (a) organic materials from the forest litter partly humified and partly in the process of being humified, and (b) some mineral soil material intermingled with the organic material mentioned and with the shallow roots of the herbaceous plants on the forest floor. This layer, or horizon, designated as the A_0 horizon is accumulative in character and is known as the humus-decay-accumulative horizon. Indeed, the leaves, twigs, and other residues from the trees, the bodies, and the roots of the herbaceous plants undergo decomposition and partly remain there, making up the volume of this horizon. It is also accumulative in another sense—the roots bring up some mineral substances from the horizons below; these are translocated to the plants, which give them up in the mineralization process of the organic matter. These mineral substances become partly fixed with the humus materials, and with the microbial flora which is instrumental in the decomposition of the organic matter, and they are partly leached downward. In this manner horizon A_0 is genetically related to the other horizons in the soil profile.

The horizon below, A_1, is usually yellowish brown in color, except in the soils that have red shale as the parent material; in this case the A_1 horizon is reddish brown. There is also some exception in color in the sandy soils in the southern part of the State in the Coastal Plain region. There the color of the A_1 horizon blends into a gray or white, and under such conditions this layer consists of bleached SiO_2, some of which is in very fine state of division, approaching at times the state of dust. In general, however, the yellowish-brown shade prevails.

This horizon is subject to the action of the decomposition products of the A_0 horizon, such as carbonic acid from respiration of roots and microbial decomposition of organic materials; nitric acid from the process of nitrification; some sulphuric acid from the process of oxidation of the sulphur compounds; and some organic acid from the decomposition of the organic materials. All of these acids leach this horizon and deprive it of its bases, its iron, and its aluminum, leaving behind some silica. The structure becomes less apparent; it becomes more powdery with fine porosity. With the downward movement of the moisture and the substances in it there is a tendency for some of the fine particles of clay also to move. The fine particles of organic materials are also caught in this downward movement.

This horizon is followed by another one, A_2, which is slightly lighter in color than the one over it. It also suffers the leaching reactions of the A_1 horizon. It gives up its bases, becomes acid, and loses some of its fine particles.

The lighter the texture of the parent material, the deeper are the horizons A_1 and A_2; and conversely, the heavier the texture, the shallower are these horizons.

Because of the losses which the A horizon (A_1 and A_2) suffers, it is known as the horizon of eluviation (washing out). Its depth varies from 15 to 60 centimeters. This horizon loses its bases and becomes enriched with silica. Some amorphous silica moves downward.

The materials which are leached out from the A horizon and are moved chemically and mechanically, are caught in the next horizon, which is known as the horizon of accumulation, or illuviation (washing in). It is designated as the B horizon. In it the fine clay particles, the iron and aluminum precipitates, and the finely divided humus materials make up a compact layer which at times becomes so cemented that it is impervious to water. Then it is known as ortstein formation, or hardpan. The depth of this horizon varies in the soils of New Jersey from 15 to 40 centimeters.

It is this layer which should primarily interest the highway engineer. Wherever an impervious layer B is formed—and it is in the more sandy soils that such a layer is likely to occur—this moisture condition, upon which the stability of the substance depends, is the most important consideration. Because of its fine texture this horizon has a tendency to attract the moisture from the layers below (the parent material) and above. The capillary moisture in the soil profile moves not only vertically but also horizontally and usually along this layer of accumulation.

It is to be remembered that this particular imperviousness exists in this zonal type of soil formation. In the pedocals there is an accumulation of calcium carbonate or sulphate, which serves as a flocculating agent, and hence the B horizon under such conditions is not impervious. The use of calcium carbonate or gypsum might be suggested in highway construction, whenever the subgrade consists of the B horizon which approaches a condition of orstein or hardpan. Such treatment might flocculate the colloids and make the horizon pervious.

In places where the water table is high, the B horizon becomes enriched with substances from the ground waters and as a result is sticky and mottled. A subgrade of this kind gives poor drainage. Whenever such a B horizon is encountered—and it may be recognized by its grayish and sometimes bluish color, occasionally with streaks of brown—it is a sure indication of a high-water table at some time during the year. Such an accumulation is known as gley. Its distinctive character is that iron compounds, because of poor aeration, are reduced. The state of oxidation of the iron compounds, as expressed by the coloration, is a true indication of the amount of gleying process this layer is subjected to. In other words, from the color one might infer the frequency and time duration of the ground waters coming up to this horizon.

Below the B horizon comes the parent material which is very little, if any, affected by the soil formers which go to make up the soil body with its characteristic profile. The parent material is usually designated by the letter C. Beyond a certain depth from the surface of the C horizon the soil scientist looks on the material as geologic formations, and the subgrade properties of the C horizon are to be elucidated more by geologic data than by soil data. The highway engineer should apply his constants to this material just the same as to any other material, be it sand, clay, or peat.

HIGHWAY ENGINEERS SHOULD MAKE USE OF SOIL PROFILES

There are a few other points which might be mentioned in connection with the possible aid a soil profile study could offer the highway engineer. In the well-known podzol zone type of soils there is a horizon of

white coloration, without structure, fine porosity, sticky and smeary when wet, but powdery and almost dusty when dry. A layer of this kind has no stability and if one occurs close to the grade line it should be taken out and wasted. Such a layer is easily recognized by a trained soils man.

It is to be remembered that the border lines of the horizons in the soil profile, although very distinct, do not run parallel to one another. The microrelief of the land will to a certain degree undulate the contours; that is, the horizon will not be alike in thickness throughout. In fixing the grade line of the road, this condition should be considered. It is often possible to fix the grade line well above or below unstable material.

Speaking of the relation of the soil properties to highway engineering one could not fail to note the possible effect of shrinkage of peat materials when lime is added. Cases are known where a fill on peat has within 15 years subsided an amount equal to 15 to 25 per cent of its original depth. Although the engineer undoubtedly takes into consideration shrinking values—and in this respect his constants are in a way determined by the chemical and physical properties of the material, whether soil material or some other material—he does not consider the possibility of chemical or biological action after the road has been put down. The peat section coming in contact with the lime leached out from the roadbed might easily become active biologically, and shrinkage would take place, a consideration unforeseen by the highway engineer.

Heaving is one of the evils in which both the highway engineer and the soils man are interested. Heaving is controlled to a certain extent by the pore space. Heavier soils with a fine porosity heave the most and flocculating the clay increases the size of the pores aggregating the colloids. At this point it is well to remember the volume relations of soils when saturated with one cation or another. Thus a soil saturated with one cation will have a different volume than when saturated with another. The introduction of certain materials in the fill might influence the cation relations in the subgrade horizon.

In laying a grade line, the engineer under certain conditions of relief could follow the compact *B* horizon—of course having determined at first its adaptability as a subgrade—and in some cases where cuts are made just to get fill material it would perhaps be better not to touch the *B* layer and to borrow fill material from the adjoining land. The profile constitution survey along the projected road would tell the engineer a good deal in respect to the depth one should excavate for the best subgrade as it is found in nature.

In soils with sand as the parent material the profile is deep, hence the *B* horizon of accumulation is located deeply. For a sand-clay road where clay is essential as a binder one would have to dig to some depth in such a sandy soil to obtain the necessary clay binder. A knowledge of soils would indicate the presence of a *B* horizon with a higher clay content than at the surface.

In the field operations many other points of contact between the pedologist and the highway engineer will come up, and their cooperative efforts will undoubtedly benefit the science and the practices of highway engineering.

(Continued from page 68)

effects of frost action on highways, procedure must be adopted which will accomplish the following purposes:

1. Prevent free water from entering the subgrade either through the surface or through seepage veins.
2. Rapidly remove free water which may be liberated during thaws.
3. Where the subgrade is lower than the adjacent ground surface lower the ground water table to an extent which will prevent harmful frost action.
4. Where other methods would not be wholly effective, remove subgrade material possessing detrimental properties to the depth of frost penetration.

BIBLIOGRAPHY

1. Density and Volume of Water. Smithsonian Phys. Tables, p. 120 (1921).
2. Watson, William. Change of Volume During Fusion. His textbook of physics, p. 236 (1911).
3. Bridgman, P. W. Effect of Pressure on the Freezing Point of Water. Smithsonian Phys. Tables, p. 200 (1921).
4. Williams, Major. Experiments on Change of Volume on Freezing Éléments de Physique, by Adolph Ganot. English translation, 18th ed., by Atkinson, E. Pp. 370–371. Idem, 14th ed., pp. 327–328.
5. Barnes, Howard T. Ice Engineering (1928). Montreal: Renouf Pub. Co. (1928).
6. Taber, Stephen. Frost Heave. Journal of Geology, vol. 37, no. 5, July–August, 1929.
7. Bouyoucos, George J. Movement of Soil Moisture from Small Capillaries to the Large Capillaries of Soil upon Freezing. Jour. Agr. Research, vol. 24, no. 5, 1923.
8. Eakin, Henry M. The Yukon-Koyukuk Region, Alaska. U. S. Geol. Survey Bul. 631 (1916).
9. Buckley, E. R. Ice Ramparts. Trans. Wis. Acad. Sci. Arts and Letters, vol. 13, pt. 1 (1900).
10. Gilbert, G. K. Lake Bonneville. U. S. Geol. Survey Monograph No. 1 (1890).
11. Nikiforoff, Constantin. Perpetually Frozen Subsoil in Siberia. Soil Science, vol. 26, no. 1, pp. 61–81.
12. Leffingwell, Ernest de K. The Canning River Region, Northern Alaska. U. S. G. S. Professional Paper 109 (1919).
13. Hobbs, William Herbert. Soil Flow. Am. Geographical Society Bul. 45, pp. 281–284. (1913).
14. Anderson, J. G. Solifluction, a Component of Subaerial Denudation. Jour. Geo., vol. 14, pp. 91–112. (1906).
15. Ekblaw, W. Elmer. Importance of Nivation as an Erosive Factor and of Soil Flow as a Transporting Agency in Northern Greenland. Nat. Ac. Sc., pt. 4, pp. 288–293. (1918).
16. Arnold, Frank P. Frost Breaks in Macadam Roads Due to Inadequate Drainage. Eng. News-Record, vol. 29, no. 20, Nov. 15, 1917.
17. Moffitt, Fred H. Geology of the Nome and Grand Central Quadrangles, Alaska. U. S. G. S. Bul. 533.
18. Tyrrell, J. B. Crystophenes or Buried Sheets of Ice in the Tundras of North America. Jour. Geol., vol. 12, p. 236. (1904).
19. Maddren, A. G. Smithsonian Exploration in Alaska in 1904 in Search of Mammoth and Other Fossil Remains. Smithsonian Miscellaneous Collection, vol. 49, pp. 44–45.
20. Buetow, W. C. Causes and Control of Damaging Frost Action in Shoulders and Subgrades in Addresses, Papers, and Discussions. Eighth Annual Asphalt Paving Conference, 1929.

(Continued from page 76)

at this time are in better condition than in January, 1929.

The behavior of section 3 especially, which has the worst appearance of any of the six experiments, makes it difficult to decide if this pavement has reached its service limit. The section as a whole is broken into a great many slabs of varying sizes and shapes, many of which are not more than 1 foot in area. In filling the cracks it is practically impossible to avoid leaving a slight ridge with a corresponding reduction in surface smoothness. However, to maintain as much smoothness as possible, some of the small areas have been entirely covered with a bituminous patch. This procedure is illustrated in Figure 6.

ASPHALTIC CONCRETE AND BRICK SECTIONS CONTINUE IN GOOD CONDITION

Maintenance of the two asphaltic concrete sections consisted of patching depressions, mostly along the west gutter line. Section 1, south of Rosemary Street, had become quite rough on the west side for a width of approximately 7 feet. It was first patched and then given a light treatment of cold tar and torpedo sand. These repairs were necessitated by the failure of the concrete base which, as stated in the previous report, had, in some areas, disintegrated to such an extent that it could be removed with shovels. This failure was more extensive on section 1 than on section 2. The surfaces of both sections are in very good condition at the present time.

The brick section, section 6, remains in excellent condition and shows little wear and no indication of failure due to the character of the surface. As shown in the preceding report, the concrete base of this experiment, excepting subsections K, L, M, and N, is in very good condition. Subsections K, L, M, and N lie on a fill which has settled and still continues to settle. Maintenance of these sections of the experiment has been high, but as it is not properly chargeable to the experiment it has been omitted from the table and curves and only the sections not affected by the fill are included.

O

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
463D. Earth, Sand-Clay, and Gravel Roads.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
1216D. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.
1486D. Highway Bridge Location.

* Department supply exhausted.

DEPARTMENT CIRCULARS

No. 94C. T. N. T. as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.
*93M. Direct Production Costs of Broken Stone. 25c.
*109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails. 10c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. 914Y. Highways and Highway Transportation.
937Y. Miscellaneous Agricultural Statistics.
1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Connecticut.
Report of a Survey of Transportation on the State Highway System of Ohio.
Report of a Survey of Transportation on the State Highways of Vermont.
Report of a Survey of Transportation on the State Highways of New Hampshire.
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio.
Report of a Survey of Transportation on the State Highways of Pennsylvania.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

MAY 31,1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
Alabama	2,140.5	$ 2,244,975.74	$ 1,109,342.18	70.6	22.1	92.7	$ 348,879.14	$ 172,648.16	25.9	0.1	26.0	$ 4,043,176.79	Alabama
Arizona	815.7	4,023,428.10	3,015,643.32	121.7	144.0	265.7	396,370.27	298,427.15	15.8	19.2	35.0	1,803,346.95	Arizona
Arkansas	1,736.4	5,518,596.26	2,571,866.18	140.2	33.5	173.7	1,041,160.03	418,357.47	56.4	21.4	78.8	2,131,678.14	Arkansas
California	1,913.9	6,723,559.18	2,566,091.28	123.5	30.1	153.6	1,120,592.67	589,515.70	39.3	8.0	47.3	2,508,229.83	California
Colorado	1,179.6	4,988,097.91	2,631,197.67	198.6	38.7	237.3	982,180.26	503,492.17	25.7	38.1	63.8	2,613,865.77	Colorado
Connecticut	240.4	2,271,591.32	981,371.99	10.7		10.7	608,889.83	163,215.00	10.9		10.9	778,000.92	Connecticut
Delaware	258.2	833,182.60	323,794.82	23.5		23.5	753,232.15	373,047.20	39.6		39.6	114,750.47	Delaware
Florida	503.5	4,924,091.30	2,257,258.51	97.1	5.5	102.6						1,782,889.95	Florida
Georgia	2,735.6	1,371,094.82	658,100.32	64.9		64.9	3,048,459.42	1,389,865.91	60.5	69.2	129.7	3,730,212.86	Georgia
Idaho	1,193.9	993,200.97	599,624.63	80.0	27.8	107.8	411,317.06	227,576.25	13.6	2.0	15.6	1,682,623.69	Idaho
Illinois	2,048.9	15,084,983.13	6,626,471.25	421.4		421.4	5,023,125.17	2,161,638.01	121.2	45.7	166.9	5,820,179.65	Illinois
Indiana	1,372.6	7,964,061.02	3,751,577.52	248.9		248.9	659,647.13	329,823.56	23.1		23.1	2,419,789.39	Indiana
Iowa	3,024.6	5,278,276.08	2,279,591.15	48.6	132.0	180.6	2,991,541.40	1,231,738.70	29.3	72.7	102.0	1,892.77	Iowa
Kansas	2,828.6	5,122,424.82	2,430,583.03	258.5	19.8	278.3	735,611.38	336,434.71	13.0	24.8	37.8	2,419,291.58	Kansas
Kentucky	1,452.1	3,855,007.56	1,821,064.33	197.2	4.2	201.4						2,814,563.49	Kentucky
Louisiana	1,355.7	4,563,418.03	2,235,066.64	142.9	12.5	155.4	706,953.08	335,603.31	20.9	4.9	25.8	1,892,079.68	Louisiana
Maine	521.5	1,828,390.73	714,287.56	46.0		46.0	1,911,044.89	642,651.39	46.9	1.6	48.5	1,382,176.03	Maine
Maryland	634.8	1,528,340.06	728,547.42	46.9	8.9	55.8	1,052,157.46	492,683.57	31.4	3.7	35.1	237,047.93	Maryland
Massachusetts	657.6	4,351,357.46	1,373,616.94	65.8	2.6	68.4	38,739.25	16,194.60	.2		.2	2,357,062.18	Massachusetts
Michigan	1,612.1	10,170,007.37	4,310,883.95	222.2	30.5	252.7	552,676.17	255,970.00	21.4		21.4	3,096,496.46	Michigan
Minnesota	3,976.0	10,196,442.72	3,586,778.31	240.6	207.9	448.5	1,823,354.82	722,175.40	11.1	61.8	72.9	24,716.29	Minnesota
Mississippi	1,810.5	1,846,209.28	716,929.45	65.6	7.7	73.3	48,835.05	24,417.52	.1		.1	3,527,874.48	Mississippi
Missouri	2,469.0	6,500,765.60	2,371,520.01	91.4	64.1	155.5	6,140,108.71	2,088,861.06	103.5	49.8	153.3	790,975.38	Missouri
Montana	1,705.5	7,870,639.46	4,591,486.48	528.2	29.7	557.9	1,398,987.89	739,242.42	113.8	50.7	164.5	3,505,060.50	Montana
Nebraska	3,639.5	7,378,837.40	3,484,632.27	314.2	133.1	447.3	1,471,193.48	584,268.63	46.8	75.0	121.8	2,476,362.21	Nebraska
Nevada	1,200.0	914,357.10	812,406.23	53.4	87.8	141.2	364,874.35	324,190.82		50.2	50.2	1,030,357.56	Nevada
New Hampshire	350.6	1,080,267.65	347,551.06	19.0	2.1	21.1						456,876.42	New Hampshire
New Jersey	504.8	6,087,677.22	1,520,866.32	69.7		69.7						977,173.76	New Jersey
New Mexico	1,886.4	4,105,389.92	2,687,028.09	203.7	50.6	254.3	318,209.24	296,765.07	16.1		16.1	1,141,623.34	New Mexico
New York	2,463.2	19,148,817.76	3,906,005.00	261.1		261.1	8,885,750.00	1,617,600.00	108.0		108.0	7,930,830.36	New York
North Carolina	1,769.3	2,886,215.94	1,425,331.25	164.9	22.6	187.5	1,254,248.73	601,459.11	17.9	5.0	22.9	2,719,753.11	North Carolina
North Dakota	4,168.3	2,207,451.13	1,029,237.21	411.4	121.8	533.2	1,142,199.07	564,294.49	132.4	232.7	365.1	1,710,232.94	North Dakota
Ohio	2,180.7	15,477,502.95	4,846,972.65	276.2	18.8	295.0	8,523,045.84	2,824,946.56	145.4	31.2	176.6	1,965,113.48	Ohio
Oklahoma	1,894.3	3,204,617.69	1,400,538.15	95.3	40.1	135.4	2,667,096.43	1,190,362.64	71.9	42.5	114.4	913,420.10	Oklahoma
Oregon	1,172.7	3,550,316.69	2,130,614.53	168.3	61.9	230.2	2,176,969.99	1,255,088.02	61.6	23.4	85.0	554,857.14	Oregon
Pennsylvania	2,287.4	17,885,222.29	4,381,259.92	256.6	14.1	270.7	4,402,365.86	1,728,902.74	40.1		40.1	2,437,641.70	Pennsylvania
Rhode Island	184.8	1,827,490.56	546,833.18	27.8		27.8	129,559.02	64,779.50	.4		.4	645,410.42	Rhode Island
South Carolina	1,898.3	3.701,357.29	1,419,947.28	104.6	24.0	128.6	3,799,401.24	1,474,579.71	45.8	99.3	145.1	210,291.41	South Carolina
South Dakota	3,437.0	4,396,141.58	2,373,749.57	471.7	124.0	595.7	585,051.42	299,930.53	50.8	56.8	107.6	1,266,452.80	South Dakota
Tennessee	1,230.8	2,734,625.44	1,248,529.22	103.8	12.5	116.3	2,064,076.74	794,392.42	61.4	27.6	89.0	2,350,474.70	Tennessee
Texas	6,840.5	11,213,047.76	4,499,483.24	374.7	72.2	446.9	4,095,666.93	1,606,742.26	107.8	79.5	187.3	5,401,577.20	Texas
Utah	988.9	1,052,178.37	724,457.39	49.7		49.7	652,094.60	476,192.35	38.5	89.6	128.1	799,257.24	Utah
Vermont	255.2	2,111,448.15	746,478.72	37.4	2.5	39.9	409,314.86	125,441.63	7.4		7.4	12,427.09	Vermont
Virginia	1,427.3	4,832,336.44	2,285,539.06	198.4	25.5	223.9	891,195.68	437,571.24	33.4		33.4	642,406.39	Virginia
Washington	933.3	3,350,076.10	1,344,800.00	68.1	40.3	108.4	802,420.05	452,600.00	30.1	6.5	36.6	1,325,573.29	Washington
West Virginia	725.2	2,922,786.34	1,182,192.93	70.5	12.5	83.0	1,162,743.13	373,570.86	11.7	28.1	39.8	811,094.06	West Virginia
Wisconsin	2,244.6	5,261,613.82	2,100,385.13	158.2	22.4	180.6	2,858,720.65	1,210,275.00	60.8	31.5	92.3	1,197,921.57	Wisconsin
Wyoming	1,735.0	1,507,919.59	1,029,715.25	138.8	46.4	185.2	502,920.49	329,810.55	31.3	76.5	107.8	1,083,530.09	Wyoming
Hawaii	41.2	853,565.90	359,459.43	21.5		21.5						1,682,039.16	Hawaii
TOTALS	[illegible]	249,813,[illegible].57	102,086,699.02	7,674.0	1,756.8	9,430.8	80,941,981.00	32,207,313.39	1,942.2	1,429.1	3,371.3	93,220,668.72	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE: 1930